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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Volume 30

April 1958-February 1960

U.S. DEPARTMENT OF COMMERCE

BUREAU OF PUBLIC ROADS

Contents of Volume 30

No. 1, April 1958:

Page

Performance of doweled joints under repetitive loading, by Leslie W. Teller and Harry D. Cashell 1

Errata (*see vol. 29, No. 11, p. 256*) 24

No. 2, June 1958:

New methods for determining capacity of rural roads in mountainous terrain, by O. K. Normann, J. O. Granum, and H. C. Schwender 25

The economic costs of motor-vehicle accidents of different types, by Robie Dunman 40

Use of the Swiss hammer for estimating the compressive strength of hardened concrete, by William E. Grieb 45

Wind tunnel for aerodynamic testing of section models of suspension bridges 51

Power shovel productivity: a motion picture 52

No. 3, August 1958:

Pore pressures in base courses, by Edward S. Barber and George P. Steffens 53

A rapid method utilizing surface area measurements in predicting the amount of cement needed to stabilize plastic soils, by Sidney Diamond and Earl B. Kinter 63

Lateral placements of trucks on two-lane highways and four-lane divided highways, by Asriel Taragin 71

No. 4, October 1958:

Evaluating trip forecasting methods with an electronic computer, by Glenn E. Brokke and William L. Mertz 77

Factors affecting trip generation of residential land-use areas, by G. B. Sharpe, W. G. Hansen and L. B. Hamner .. 88

Table: State legal maximum limits of motor vehicle sizes and weights compared with AASHO standards 89

Current structural bridge steels: a survey of usage and economy, by Nathan W. Morgan 100

No. 5, December 1958:

Travel patterns in 50 cities, by Frank B. Curran and Joseph T. Stegmaier 105

Observations concerning urban traffic volume patterns in Tennessee, by Boris B. Petroff and Anthony P. Kancler .. 123

Traffic article postponed 131

No. 6, February 1959:

Progressive alterations in a sheet asphalt pavement over a long period of service, by Jarl T. Pauls and Woodrow J. Halstead 133

Airphoto analysis of terrain for highway location studies in Maine, by Ernest G. Stoeckeler and William R. Gorrill .. 145

No. 7, April 1959:

Trends in travel to the central business district by residents of the Washington, D.C., metropolitan area, 1948 and 1955, by Jacob Silver 153

Lost mixing time of dual drum pavers: a motion picture 176

No. 8, June 1959:

Page

Correlation between chemical and mortar bar tests for potential alkali reactivity of concrete aggregates, by Bernard Chaiken and Woodrow J. Halstead 177

Home-interview traffic surveys and related research activities, by John T. Lynch 185

Pretreatment of soils and clays for measuring external surface area by glycerol retention, by Earl B. Kinter and Sidney Diamond 187

Nonelastic deformations in continuous concrete structures, by Murrel O. Wilburn 191

No. 9, August 1959:

Properties of highway asphalts—Part 1, 85–100 penetration grade, by J. York Welborn and Woodrow J. Halstead 197

Map: Maximum wind speeds to consider in designing highway signs 208

North Dakota's use of aerial inventory for county general highway maps, by E. T. Bowen, C. J. Crawford, and J. B. Kemp 210

Characteristics of traffic entering and leaving the central business district, by David A. Gorman and Stedman T. Hitchcock 213

No. 10, October 1959:

The use of backwater in the design of bridge waterways, by Joseph N. Bradley 221

Assigning traffic to a highway network, by Glenn E. Brokke .. 227

Vehicle acceptance rates of parking areas, by Arthur A. Carter, Jr., and Bessie A. Hayes 232

Traffic signals and accidents in Michigan, by David Solomon . 234

Two simple techniques for determining the significance of accident-reducing measures, by Richard M. Michaels .. 238

Estimated travel by motor vehicles in the United States, 1957, by Alexander French 240

No. 11, December 1959:

Circular reinforced concrete columns subjected to direct stress and bending, by A. Anthony Toprac 241

Sampling techniques applicable to the collection of economic data, by Nathan Lieder 246

Table: State legal maximum limits of motor vehicle sizes and weights compared with AASHO standards 250

Factors influencing mass-transit and automobile travel in urban areas, by Warren T. Adams 256

No. 12, February 1960:

Forecasts of population, motor-vehicle registrations, travel, and fuel consumption, by Thomas R. Todd 261

Estimated travel by motor vehicles in the United States, 1958, by Alexander French 275

Common-carrier passenger and freight services available to communities on the Interstate Highway System, by Arthur K. Branham and Florence K. Banks 276

Surface and subsurface temperature variations and comparisons, by Harold L. Boen and Gerard A. DeMarrais .. 283

List of Authors

(*and volume page references*)

	Page		Page		Page		Page
Adams, W. T.	256	DeMarrais, G. A.	283	Hitchcock, S. T.	213	Schwender, H. C.	25
Banks, F. K.	276	Diamond, Sidney	63, 187	Kancler, A. P.	123	Sharpe, G. B.	88
Boen, H. L.	283	Dunman, Robie	10	Kemp, J. B.	210	Silver, Jacob	153
Bowen, E. T.	210	French, Alexander	240, 275	Kinter, E. B.	63, 187	Solomon, David	234
Bradley, J. N.	221	Gorman, D. A.	213	Lieder, Nathan	246	Steffens, G. P.	53
Branham, A. K	276	Gorrill, W. R.	145	Lynch, J. T.	185	Stegmaier, J. T	105
Brokke, G. E.	77, 227	Granum, J. O.	25	Mertz, W. L.	77	Stoeckeler, E. G.	145
Carter, A. A., Jr.	232	Grieb, W. E.	45	Michaels, R. M.	238	Taragin, Asriel	71
Cashell, H. D.	1	Halstead, W. J.	133, 177, 197	Morgan, N. W.	100	Teller, L. W.	1
Chaiken, Bernard	177	Hamner, L. B.	88	Normann, O. K.	25	Todd, T. R.	261
Crawford, C. J.	210	Hansen, W. G.	88	Pauls, J. T.	133	Toprac, A. A.	241
Curran, F. B.	105	Hayes, B. A.	232	Petroff, B. B.	123	Welborn, J. Y.	197
						Wilburn, M. O.	191

VOL. 30, NO. 1

APRIL 1958

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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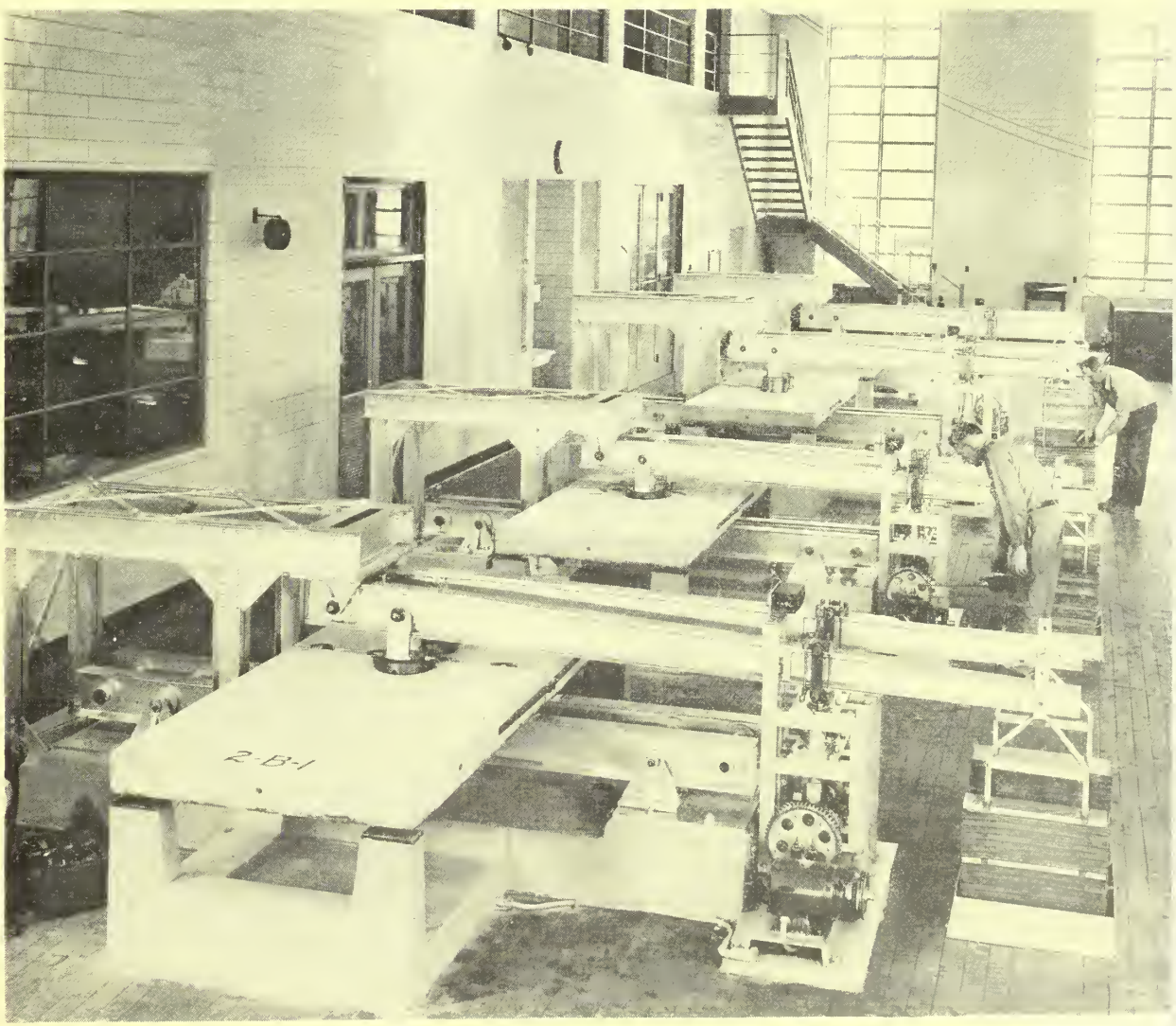

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: A discussion of laboratory tests of doweled joints in concrete pavements.

199469

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 1 *April 1958*

C. M. Billingsley, *Editor*

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Washington 25, D. C.

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IN THIS ISSUE

Performance of Doweled Joints Under Repetitive Loading 1

New Publications 24

Errata 24

PUBLIC ROADS is sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1 per year (25 cents additional for foreign mailing) or 20 cents per single copy. Free distribution is limited to public officials actually engaged in planning or constructing highways, and to instructors of highway engineering. There are no vacancies in the free list at present.

The printing of this publication has been approved by the Director of the Bureau of the Budget, Mar. 17, 1955.

U. S. DEPARTMENT OF COMMERCE
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Under Secretary for Transportation

BUREAU OF PUBLIC ROADS
BERTRAM D. TALLAMY, Administrator

Contents of this publication may be reprinted. Mention of source is requested.

Performance of Doweled Joints Under Repetitive Loading

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by LESLIE W. TELLER, Chief, Structural Research Branch, and HARRY D. CASHELL, Highway Physical Research Engineer

Load-transfer systems are desirable in transverse joints of concrete pavements to control edge stresses, to reduce slab deflections under load, and to maintain surface alinement of the two slab ends.

Many forms of testing techniques have been devised to judge the performance of load-transfer systems. In most cases the tests have been performed in the laboratory and, as usually made, provide data on the shear resistance of the load-transfer unit under a single or, at most, a few static loads. Such tests, however, are rather limited in scope.

In order to develop information on the structural action of load-transfer systems under repetitive loading, the Bureau of Public Roads devised a laboratory procedure quite different from the shear test. The principle of the test is very simple. The specimen, a concrete slab divided transversely at midlength by the joint under test, is supported in a machine that applies a known load alternately on either side of the joint for any desired number of cycles. The design of the machine and the dimensions of the specimen made it possible to study, under forces and motions which simulate closely those of actual service, the effects of several variables influencing the structural performance of dowel bars.

An analysis of the data developed in the tests revealed that a definite exponential relation exists between dowel diameter and load-transfer capacity, other conditions being constant.

A relation was also evident between slab depth and the dowel diameter required to transfer a given percentage of the applied load. This relation indicated that, for minimum dowel size, the diameter in eighths of an inch should approximately equal the slab depth in inches.

For ¾-inch diameter dowels, an embedded length of 8-dowel diameters is required for maximum load transfer. Larger dowels, such as the 1-inch and 1¼-inch diameters now in common use, require for full-load transfer a length of embedment of about 6 diameters, both initially and after many hundreds of thousands of cycles of repetitive loading.

For a given load-transfer system, the tests indicated that much better structural performance could be expected in a contraction joint than in an expansion joint.

THE USE of smooth, round steel bars across transverse joints in concrete pavements for the purpose of transferring load seems to have been first reported in connection with a pavement built in the winter of 1917–18 between two army camps near Newport News, Va. In this installation four ¾-ineh diameter bars were used in the 20-foot pavement width.

In the years that followed World War I, the use of steel dowels, as they came to be called, spread quite rapidly. During this period the detailed requirements as to diameter, length, and spacing varied widely however. For a joint across the full width of the pavement, one State in 1926 required two ½-inch diameter bars 4 feet long; another, four ⅝-inch diameter bars 4 feet long; and still another, eight ¾-inch diameter bars 2 feet long. Long bars of small diameter spaced about 30 inches apart were the general rule. By 1930 nearly half of the States required the use of dowels in transverse joints.

In 1928 Westergaard (*1*) [2] published the first analysis of dowel reactions. It was based on certain assumed ideal conditions including equal deflection on both sides of the joint. He concluded that dowels 2 feet apart would bring about a material reduction in critical stress in the concrete of the pavement, but at a spacing of 3 feet they would not. No study of dowel length was made.

When the designs for the pavement sections used in the Arlington tests (*2*) were being developed in 1929, it was decided to include a study of doweled joints; dowel spacings of 18, 27, or 36 inches were selected for the various transverse joints. The dowels were ¾ ineh in diameter and 3 feet long in all cases. These dimensions and spacings were representative of State practices at that time.

As a result of the load tests on the doweled joints in the Arlington investigation, it was concluded that none of the dowel systems tested was particularly effective in controlling critical edge stress and that dowels would have to be stiffer and much more closely spaced to be effective structurally.

The Bureau's researches into the structural action of concrete pavements stimulated a considerable amount of interest in the subject on the part of others. One result was an increased effort to develop a better understanding of the structural action of doweled joints and to rationalize their design.

In 1932 Bradbury (*3*) attempted to determine analytically the required diameter, length, and spacing of dowels. His studies indicated the need for larger diameter dowels at close spacing; and, through the application of the Timoshenko equations for the bending of bars on elastic foundations, he developed a formula for estimating the required length of dowels. In 1938 Friberg (*4*) analyzed the dowel reactions by means of the same equations and reported an experimental study of the support afforded dowels by the surrounding concrete. Friberg also emphasized the advantages to be gained from increasing dowel diameter and decreasing dowel spacing. He concluded that the length of dowels could be materially reduced below the 24 inches then in common use.

Westergaard (*1*) in his analytical studies of dowel reactions had concluded that the major part of the load transfer which takes place when a wheel load approaches a transverse joint is accomplished by the 2 or, at most, 4 dowels nearest the wheel load. In 1940 Kushing and Fremont (*5*) published a theoretical analysis of the distribution of reactions among the several units of a doweling system, assuming elastic deflection of the dowels. As

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] Italic numbers in parentheses refer to the list of references on page 24.

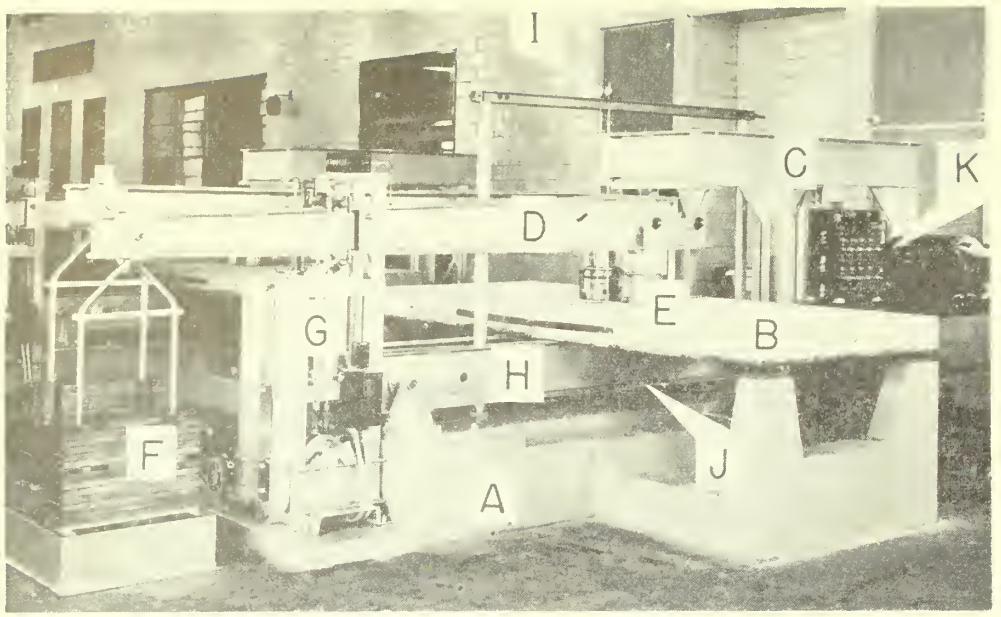

Figure 1.—Testing machine (A, reinforced concrete base; B, test specimen; C, structural steel frame; D, loading lever; E, loading pad; F, adjustable load of lead weights; G, cam and lift-rod mechanism; H, specimen support beam; I, micrometer dials; J, SR-4 strain gages; and K, oscillograph recording equipment).

would be expected, this analysis indicated a wider distribution of reactions than was indicated by Westergaard's earlier study in which the dowels were assumed to be infinitely stiff. In discussing the Kushing and Fremont paper, Sutherland presented data from load tests on certain large slabs in the Arlington investigation. These data indicated, for the conditions of the tests with ¾-inch diameter dowels at 18-inch spacing, the relative deflection of the 2 abutting slabs was largely controlled by the 4 units immediately adjacent to the load.

This brief review of research activity prior to about 1940 shows much progress in the effort to rationalize the structural design of doweled joints. The increased use and the cost of load-transfer systems made the problem of proper design an important one. The researches indicated the need for strong units closely spaced. It was also indicated that the long dowel bars used earlier were not necessary, and there was some optimum length for maximum effectiveness. The inherent structural deficiencies of the round steel dowel bar as a load-transfer mechanism had long been recognized and many alternate designs, frequently proprietary, were offered during this period. Some of these designs were simple structural shapes of greater stiffness; others were quite elaborate.

Early Test Procedures Inadequate

State highway departments and other agencies responsible for the selection or approval of competitive designs sought comparative data on which to base decisions. The need for test procedures to develop such data became pressing, and many forms of testing techniques were devised to meet the need. In nearly all cases the tests were performed in the laboratory by applying a load to a relatively small specimen in such a way as to develop a shearing force, usually on 1 but sometimes on 2 or more dowels or other types of load-transfer units embedded in concrete. The load-deflection data obtained in the tests were used in various ways for judging the relative abilities of dowels and other devices in transferring load across joint openings. A typical test procedure of this type, as performed in Illinois, is described in the report of that State's investigation of joint performance (*6*).

Finney and Fremont (*7*) used a modification of this test procedure to study the effect on dowel deflection of the variables of dowel diameter, dowel length, and width of joint opening.

The laboratory shear test, as usually made, develops data on the relative shear resistance of load-transfer units under a single or, at most, a few essentially static loads. It does not, however, provide other information of equal or greater significance.

If there is any play or looseness of the dowel in its socket or, in the case of some proprietary load-transfer units, play within the unit itself, this looseness will not be revealed in the shear test even though it would have an important bearing on the structural effectiveness of the unit when the load is reversed as it is in service.

Furthermore a dowel or other load-transfer unit in the pavement is placed in action every time an axle load crosses the transverse joint—thousands, even millions of times. Each time this happens there is a complete stress reversal in the load-transfer mechanism as the load passes from one abutting slab to the other. It is well recognized that performance under a single loading is no measure of performance under repeated loading, yet as late as 1947 there existed no published data on the effects of repetitive loading and stress reversal on the structural action of dowels or other load-transfer units.

These considerations led the Bureau of Public Roads in 1947 to devise a test procedure of quite a different type, one which would provide information on the structural action of load-transfer units under repetitive loading. It was desired particularly to determine (1) the initial efficiency of such units in transferring load; (2) the degree to which this efficiency might be expected to be retained as the load cycle is repeated many thousands of times; and (3) the effect on load-transfer efficiency of such major design variables as dowel diameter, dowel length, and width of joint opening.

It is the purpose of this report to describe the test and to discuss the information that it has so far provided.

Summary

A new machine and method of test have been developed which provide a satisfactory means for studying, under repetitive loading, the effects of the several variables which influence the structural performance of dowel bars or other load-transfer devices used in the joints of concrete pavements. The conditions of the test approach closely those which are found when a heavy wheel load crosses a transverse joint of a pavement in service.

The test procedure makes possible a determination of the initial effectiveness of a load-transfer system as well as any loss in initial

Figure 2.—Looking down on the mold in which the specimens were cast.

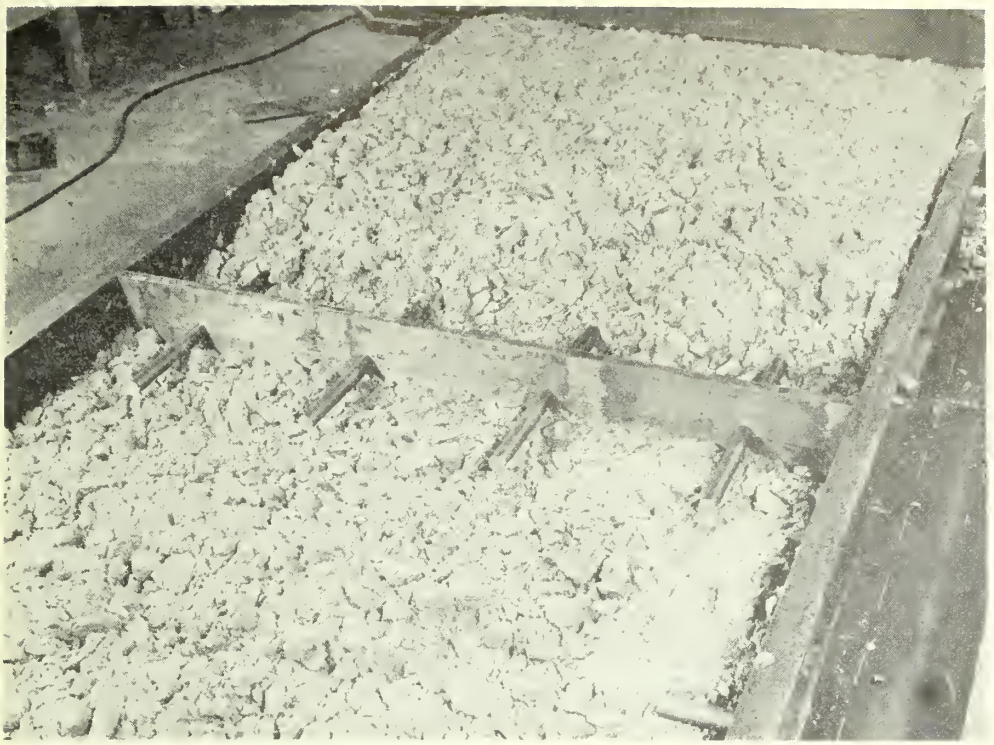

Figure 3.—Appearance of concrete during fabrication of a specimen: fluidity was obtained with internal vibrators.

effectiveness which may develop as the result of a large number of applications of the loading cycle.

With this test procedure the principal effort thus far has been to determine the influence on the structural performance of round steel dowels of three important variables: dowel diameter, length of dowel embedment, and width of joint opening. For each of these variables an orderly relation has been established.

The following conclusions are based on an analysis of the data presented in this report.

1. A definite exponential relation exists between dowel diameter and load-transfer capacity, other conditions being constant.

2. A relation exists between slab depth and the dowel diameter required to transfer a given percentage of the applied load. This relation may be expressed as an approximate rule for minimum dowel size, as follows: For round steel dowels at a 12-inch spacing in joint openings of ¾-inch width or less, the dowel diameter in eighths of an inch should equal the slab depth in inches.

3. The length of dowel embedment necessary to develop maximum load transfer is not a constant function of dowel diameter as has sometimes been assumed. With a ¾-inch dowel diameter, maximum load transfer requires an embedded length of about 8-dowel diameters. With larger dowels, such as the 1-inch and 1¼-inch diameters now in common use, full-load transfer is obtained with a length of embedment of about 6 diameters, both initially and after many hundreds of thousands of cycles of repetitive loading. The use of shorter dowels in these larger diameters would, in many cases, result in an appreciable savings in the amount of steel required for dowels.

4. For a given dowel diameter and condition of loading, decreasing the width of joint opening decreases the bending stress in the dowel. It decreases also the dowel deflection and hence increases the percentage of load transferred, both initially and after extended repetitive loading. It is evident that a given load-transfer system may be expected to give a much better structural performance in a contraction joint than in an expansion joint of ¾-inch width or greater.

5. The condition of dowel looseness has an important effect on the structural performance of the dowel, since it can function at full efficiency only after this looseness is taken up by load deflection. This is true for both initial looseness and that which develops during the course of repetitive loading. Tests which do not include repetitive loading and complete stress reversal provide no information on this important condition and no measure of its effects.

6. The application of extended repetitive loading decreases the initial ability of a given system to transfer load. Under equal conditions, the amount of this loss varies considerably as dowel diameter, length of dowel embedment, and width of joint opening are varied.

In the tests of the authorized program as described in this report, much important information was obtained on the structural performance of round steel dowels under repetitive loading. To complete the research, additional tests are needed. Recommendations as to the nature and extent of these tests are included in the report.

Testing Machine and Specimen Described

The principle of the test is very simple. The specimen, a concrete slab divided transversely at midlength by the joint under test, is supported in a machine that applies a known load alternately on either side of the joint for any desired number of cycles.

By means of strain and deflection measurements made periodically, data are obtained which show the initial effectiveness of the load-transfer system and the deterioration in effectiveness which develops from repetitive application of the test load.

Testing machine

The machine, shown in figure 1, consists essentially of a concrete base which provides support for the specimen, a structural steel frame which furnishes a reaction for a pair of loading levers that apply the load through 10-inch diameter loading pads alternately on

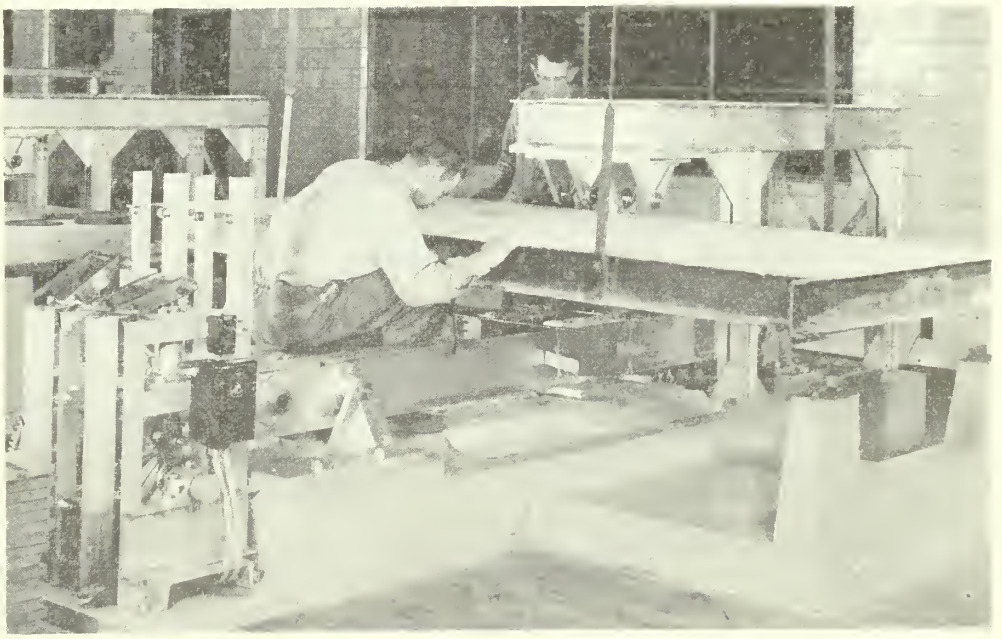

Figure 4.—Lowering a cured specimen into the testing machine in the channel frame in which it was cast; the loading levers have been removed for this operation.

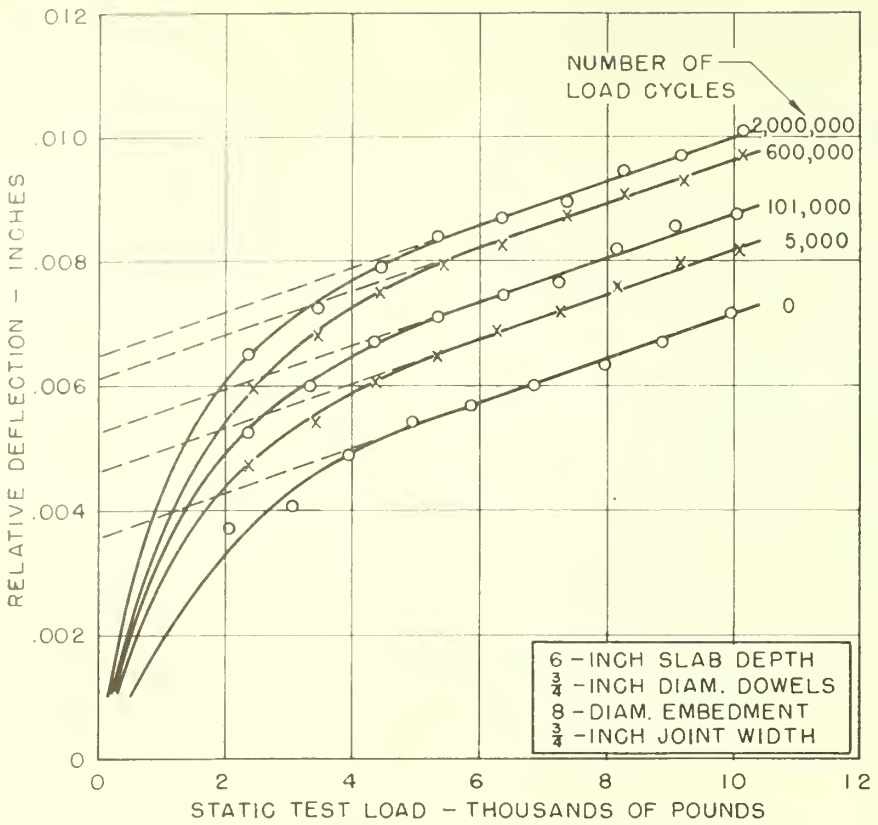

Figure 5.—Relation between applied load and relative deflection, after various numbers of load cycles.

either side of the joint under test, dead weights at the ends of the loading levers to create the desired load, and an electrically driven cam and lift-rod mechanism which alternately raises and lowers the loading levers. The machine requires very little attention and can operate continuously.

The specimen, a concrete slab 10 feet long and 4 feet wide, is supported at its ends by fixed bearings on short pedestals that are a part of the machine base. On either side of the test joint each half-slab bears on a steel support beam. The deflection of the beam simulates the yielding of the subgrade when a load is applied to a pavement slab. As the load is applied on one side of the test joint by the lowering of one loading lever, the load is automatically removed from the opposite side of the joint by the lifting of the other loading lever. As this action takes place, the support beam under the loaded side deflects from the load while the support beam under the unloaded side is deflected solely by the shearing forces in the doweling system just as in the case of a pavement on the subgrade.

Two span lengths were provided for the support beams, the relation between them being such that use of the greater span length would result in deflections twice as great as would be obtained with the lesser span length, other test conditions remaining unchanged.

The dimensions of the machine are such that the deflections and the angular motion at the joint can be made to simulate closely the movements that have been measured in the load testing of full-size pavement slabs on a weak subgrade. By adjusting the length of bearing between the specimen and the support beam directly under the load, the degree of transverse curvature of the specimen and hence the stress in the concrete along the joint edge can be controlled. For the tests reported, a bearing pad having a length of 15 inches was used. With a constant load value this gives a flexural stress in the concrete along the joint edge which varies with the depth of slab being tested.

The machine is designed to apply loads rather slowly so as to avoid shock, the frequency being but 10 cycles per minute. At this rate about one week is required to obtain 100,000 complete cycles of load application and stress reversal. After the first machine was placed in operation it became apparent that even a limited program would extend over an excessively long period of time, and since the test appeared to be promising three additional machines were built. Two of the four have a load capacity of 10,000 pounds; the other two have a capacity of 15,000 pounds. Any desired test load, up to the capacity of the machine, may be obtained by changing the number of lead weights on the platforms at the end of the loading levers.

The loading system is calibrated by placing a load-measuring device between the pad on the loading lever and the corresponding support beam before the specimen is placed in the machine. The load-strain rate of each support beam is then developed from loads applied by the calibrated loading system and from strains as measured in the lower flanges of the beams. These rates provide a means of determining the amount of load being transferred for any given applied load. Since the unit of strain measurement is equivalent to a load increment of about 30 pounds and since periodic calibrations showed negligible changes, it was concluded that determinations of the amount of load transferred were accurate within 60 pounds.

In figure 1, a pair of tubular columns supported in steel sleeves in the base and extending upward above the machine frame may be seen. They support a cross member or bridge which serves as a datum for deflection measurements. These measurements were made with micrometer dials reading directly to thousandths of an inch from which it was practicable to estimate ten-thousandths.

Test specimen

As stated previously, the test specimen is a concrete slab 4 feet wide by 10 feet long divided transversely at midlength by the joint in which the load-transfer system is installed. So far, the slabs have been either 6, 8, or 10 inches in depth.

Since the quality of the concrete has not been a variable in the test program being reported, every effort has been made to have the strength and other properties of the concrete uniform in the specimens that have been tested. The same aggregates, grading, and proportions were used throughout. The concrete was mixed under careful control in the laboratories of the Bureau. The following summary shows average strengths and other

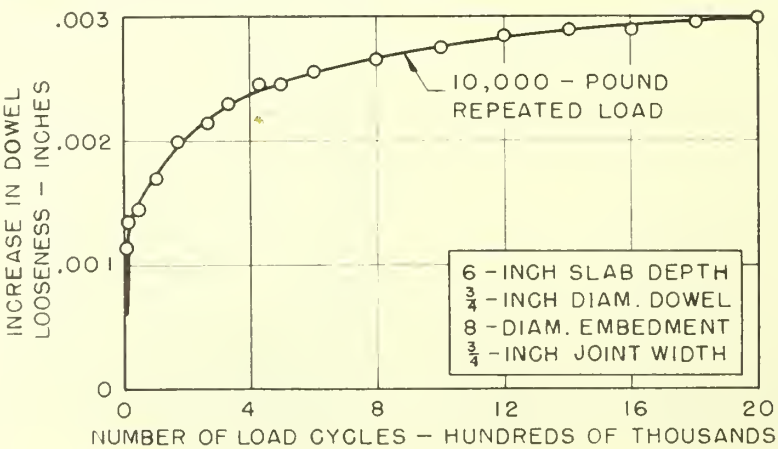

Figure 6.—Effect of repetitive loading on the development of dowel looseness.

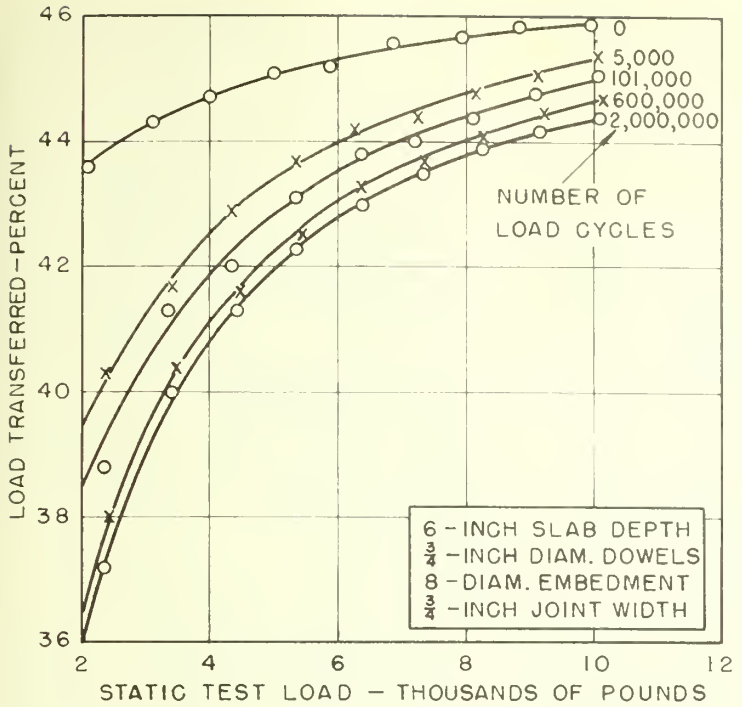

Figure 7.—Relation between applied load and percentage of load transferred, after various numbers of load cycles.

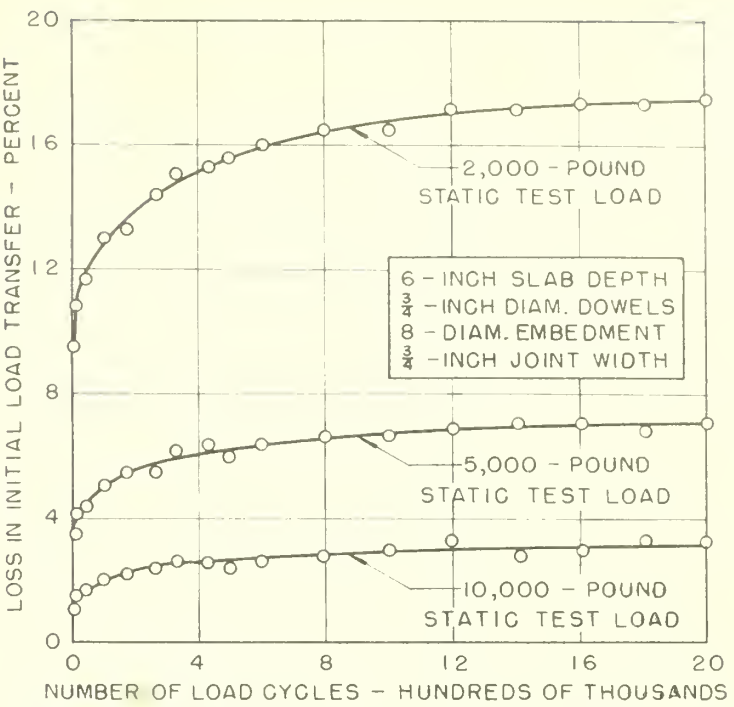
LOSS IN INITIAL LOAD TRANSFER — PERCENT
NUMBER OF LOAD CYCLES — HUNDREDS OF THOUSANDS
2,000-POUND STATIC TEST LOAD
5,000-POUND STATIC TEST LOAD
10,000-POUND STATIC TEST LOAD
6-INCH SLAB DEPTH
¾-INCH DIAM. DOWELS
8-DIAM. EMBEDMENT
¾-INCH JOINT WIDTH

Figure 8.—Loss in initial capacity to transfer load resulting from repetitive loading.

properties of the concrete as determined by standard tests at the age of 28 days: compressive strength 5,610 p. s. i., standard deviation 280 p. s. i.; flexural strength 770 p. s. i., standard deviation 35 p. s. i.; and modulus of elasticity (sonic) 7,120,000 p. s. i.

It was particularly important that the slab specimens be precision cast and so handled that the joint installation could not be damaged before the test started. To accomplish this a concrete casting base was built. In the surface of this base were steel plate inserts to create smooth, plane, parallel surfaces on the lower surface of the specimen for the points of bearing in the testing machine. On this base the side form for the specimen was placed. This was a rectangular frame of structural steel channel of the required depth, across the center of which was fastened the steel plate partition that created the joint opening and held the load-transfer units in position and alinement. The concrete was consolidated by vibration. After the surface had been finished the specimen was covered with burlap, kept wet for 7 days, and then allowed to dry in the air of the laboratory. In general, the specimen was more than 28 days old before being subjected to load.

The dowels of the load-transfer systems were made from conventional hot-rolled, carbon steel bar stock. Tests of the material showed it to have average mechanical properties, as follows:

Tensile strength	p.s.i.	66,533
Yield point	p.s.i.	44,327
Percent elongation (2-inch gage length)		40.5
Modulus of elasticity	p.s.i.	30,094,000

All dowels were so installed that their final position did not vary from true alinement by more than 1/16 inch per foot of length. Just prior to concreting, the free or sliding half of each dowel was coated with heavy oil to prevent bonding of the concrete.

Care exercised in handling specimen

Following the completion of the curing, the specimen was moved from the casting base to the testing machine in the channel frame. To hold the slab securely in the frame during this operation, short steel dowels were cast in the concrete around the perimeter of the specimen. These extended through close-fitting holes drilled in the web of the channel and supported the weight of the specimen in the frame when the latter was lifted. Figure 2 shows these details as one looks down into the casting form. Figure 3 shows a partially filled form before the vibrating of the concrete had been started, and is included to give an idea of the character of the concrete mixture. Figure 4 shows a specimen in the frame being lowered into position in the testing machine. Once the specimen was properly placed, the form was disassembled and removed together

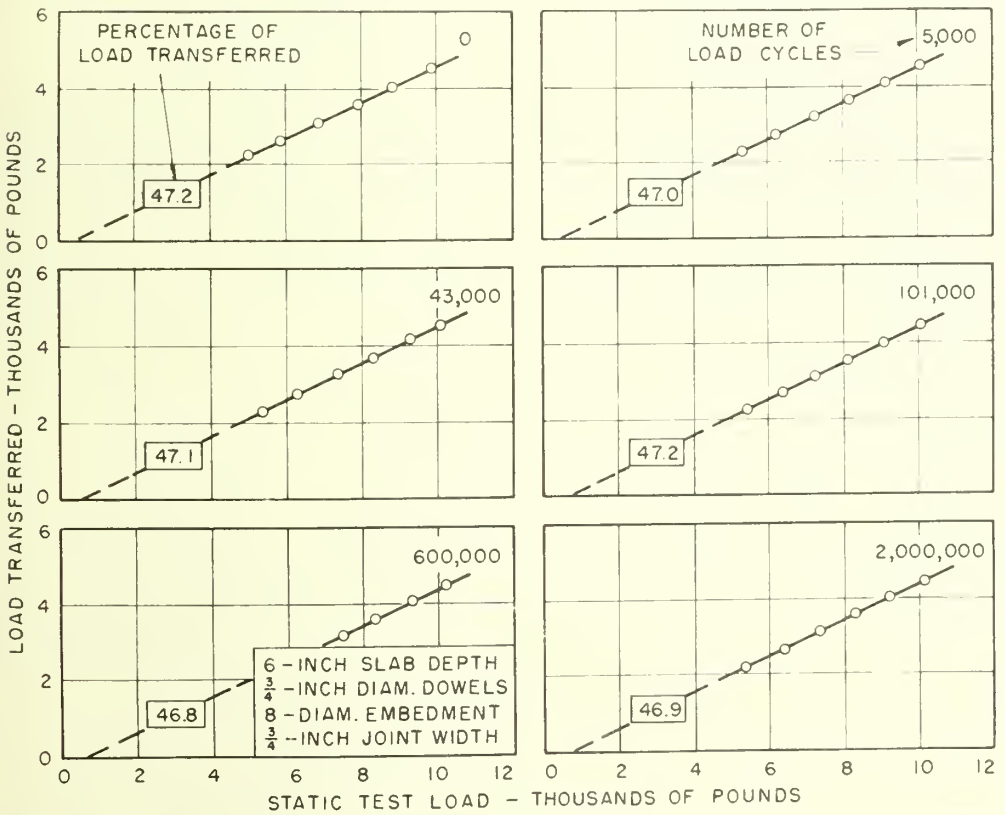

Figure 9.—Relation between applied load and amount of load transferred, after various numbers of load cycles.

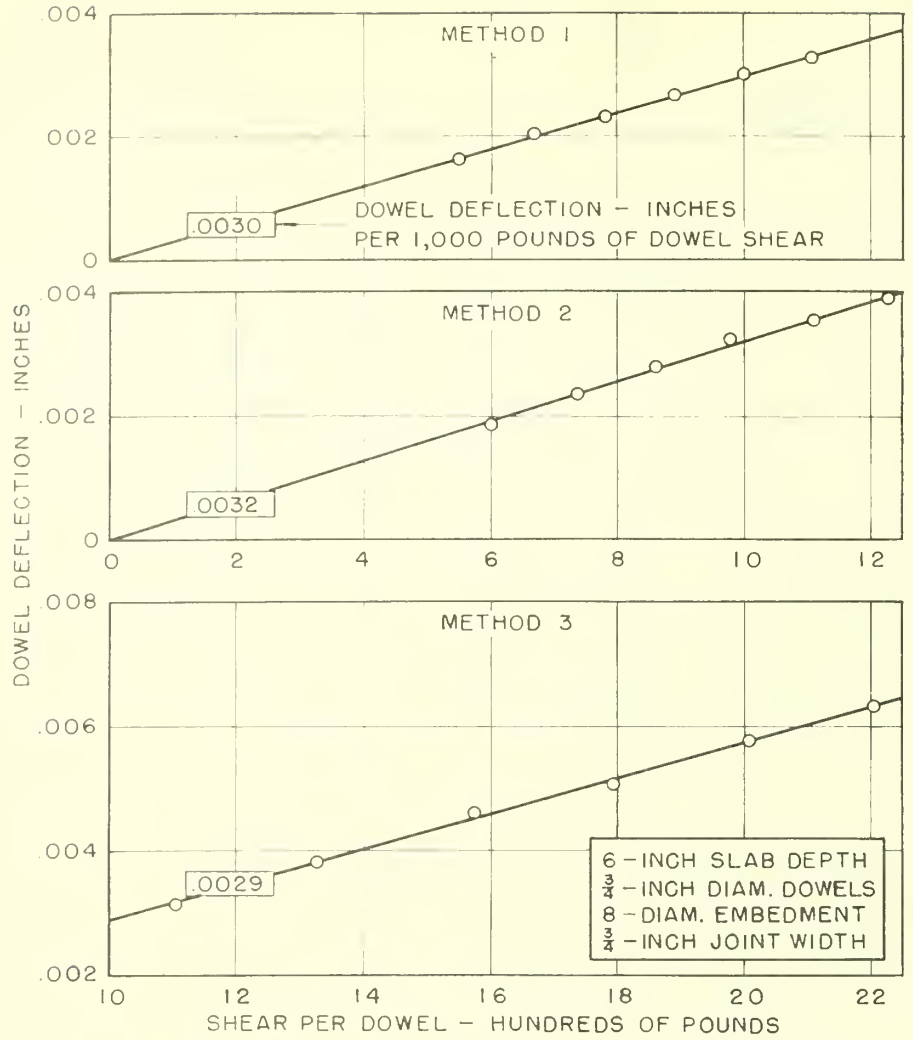

Figure 10.—Typical relations between dowel shear and dowel deflection.

with the steel partition used to create the joint opening. By this procedure all of the specimens were successfully handled and placed in the testing machines without damage to the joint system.

As stated earlier, the test was designed so as to make it possible to subject the load-transfer units under test to forces and motions that would simulate closely those which would be encountered in service. These requirements determined the general size of the specimens and the machine for testing them.

The earlier tests of full-size slabs at Arlington, Va., (*2*) had supplied data on the load-deflection relation of doweled joints in pavement slabs of several thicknesses. Data were obtained also on the angular motion that occurs when a slab end is deflected by load. This information was used to determine the dimensions of the support beams and, in turn, the length of the test specimen. Because of the indications of various analyses of dowel reactions and of the experiments referred to by Sutherland (*5*), it was decided to make the concrete specimen wide enough to permit the installation of 4 load-transfer units 12 inches apart. This led to the selection of the 48-inch width mentioned earlier. Since the experiments were expected to include load-transfer units of various sizes and strengths, the design of the machines provided for a range of loads and specimen depths. Very few concrete highway pavements have been built with thicknesses of less than 6 inches or more than 10 inches. For this reason, slab depths of 6, 8, and 10 inches were selected for study as stated previously.

From this description it is apparent that the test lends itself to studies of the structural behavior of load-transfer systems under conditions that approach those of actual service and provides a means for obtaining new and useful information on the effects of repetitive loading.

Impact conditions of load application have been purposely avoided: first, because a joint that causes appreciable impact is usually either a poorly constructed or a failed joint and, second, because impact is a complicated phenomenon difficult to control and to evaluate.

With the machine described it is possible to study any one of a number of variables that influence the structural performance of load-transfer systems. The work that has been done thus far has been largely confined to studies of the effects of the three variables of dowel diameter, dowel length, and width of joint opening. Certain collateral studies that were made for various reasons are described later.

Test Procedure Based Upon Preliminary Studies

After the completion of the first machine, it was necessary to make a number of preliminary studies to determine what the details of the test procedure should be to yield the most pertinent information in the least time. Some of the questions that needed to be answered were as follows:

1. How many cycles of loading would be required to produce significant comparisons

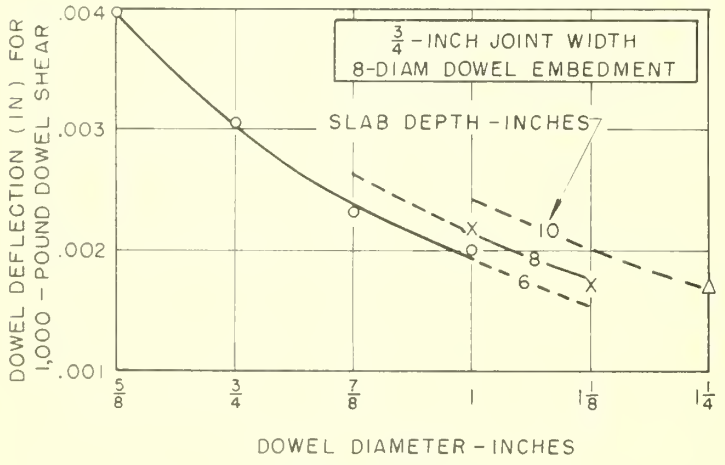

Figure 11.—Relations between dowel diameter and dowel deflection.

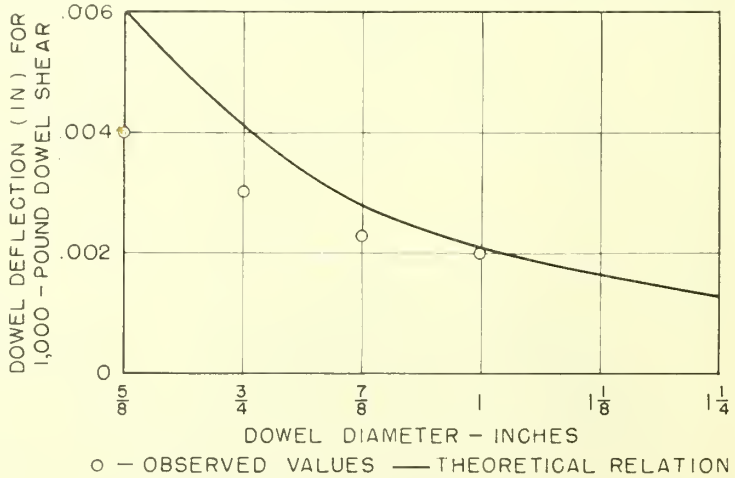

Figure 12.—Comparison of theoretical and observed relations between dowel diameter and dowel deflection.

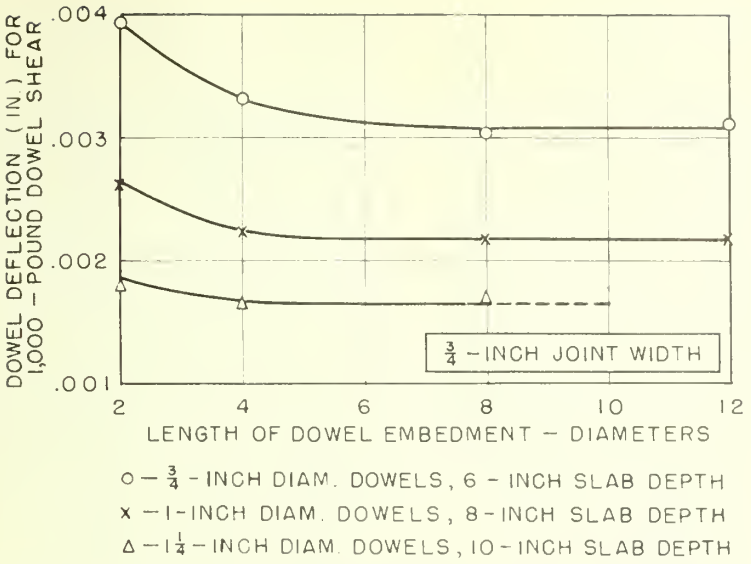
DOWEL DEFLECTION (IN.) FOR 1,000 - POUND DOWEL SHEAR
.004
.003
.002
.001
2
4
6
8
10
12
LENGTH OF DOWEL EMBEDMENT - DIAMETERS
¾ - INCH JOINT WIDTH
o - ¾ - INCH DIAM. DOWELS, 6 - INCH SLAB DEPTH
x - 1 - INCH DIAM. DOWELS, 8 - INCH SLAB DEPTH
Δ - 1¼ - INCH DIAM. DOWELS, 10 - INCH SLAB DEPTH

Figure 13.—Relations between length of dowel embedment and dowel deflection.

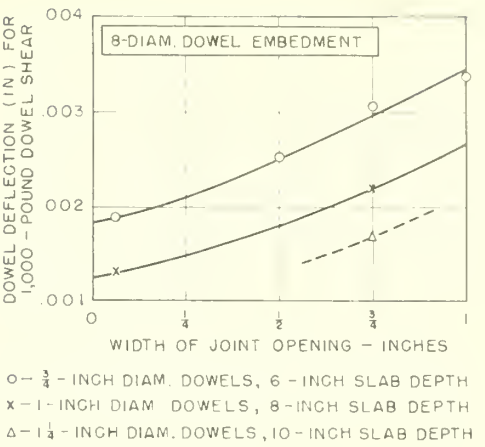
DOWEL DEFLECTION (IN.) FOR 1,000 - POUND DOWEL SHEAR
.004
.003
.002
.001
8-DIAM. DOWEL EMBEDMENT
0
¼
½
¾
1
WIDTH OF JOINT OPENING - INCHES
o - ¾ - INCH DIAM. DOWELS, 6 - INCH SLAB DEPTH
x - 1 - INCH DIAM. DOWELS, 8 - INCH SLAB DEPTH
Δ - 1¼ - INCH DIAM. DOWELS, 10 - INCH SLAB DEPTH

Figure 14.—Relations between width of joint opening and dowel deflection.

and what additional information would be obtained by continuing the test beyond this point?

2. What strain and deflection data should be obtained and at what intervals should each measurement be made?

3. With two joint deflection values available, as the result of providing two span lengths for the support beams, what would be the relative effect of each on the test results?

The determination of the number of cycles of loading needed to develop significant data is an important matter. An effort was made to ascertain from traffic survey data how many wheel loads of approximately 10,000 pounds might be applied at a given point on a transverse joint of a pavement on a heavily traveled route in the course of a year. It was found, however, that data on the transverse placement of wheel loads of this magnitude were not available, and the effort to relate the test duration to periods of pavement service was abandoned, at least for the early program. It was then decided to study the structural behavior of a few specimens under repeated loading; and, on the basis of these observations, make a decision on the question of test duration and others related to the test procedure.

When a load is applied on one side of the joint under test, a deflection develops in the support beam directly under the load and, depending upon the characteristics of the load-transfer system, in the companion support beam on the other side of the joint also. If there were no elastic deformation in either the steel dowels or the concrete and if there were no looseness or play to be taken up, the two abutting slab edges would move downward by an equal amount and the two support beams would deflect equally. Under this ideal condition, the amount of load transferred across the joint would be one-half of that being applied to the surface of the test specimen.

Elastic deformation in both dowel and concrete

Actually there is elastic deformation in both the dowel steel and the concrete, and even in these carefully constructed laboratory specimens there is some initial looseness in the seating of the dowel in its socket, although in these tests its magnitude was small as will be shown later. The result is that the amount of load transferred was always less initially than the 50-percent theoretical maximum and tended to decrease as the loadings were repeated.

As an important part of the preliminary studies, a representative specimen was subjected to a test in which a 10,000-pound load was applied alternately on either side of the joint 2 million times.

The specimen in this case was a slab 6 inches in depth; the joint opening was ¾ inch; the 4 dowels were ¾-inch diameter; and the length of their embedment in the concrete on either side of the joint opening was 8-dowel diameters. The lesser span length of the support beams was used which resulted in a midspan deflection rate of 0.01 inch per 1,000 pounds of applied load, assuming no load transfer across the joint.

The purpose of the test was primarily to develop general information required for the detailed planning of the testing procedure and the test program. As the loading cycle was repeated on this specimen, the program was interrupted at frequent intervals in order that measurements might be made of strains in the lower flanges of the support beams and deflections of the slab surfaces on either side of the joint opening. These measurements were made under a sequence of statically applied loads, the magnitude of which was varied from 2,000 to 10,000 pounds by 1,000-pound increments.

From the differences in strain and the differences in deflection measured on the loaded and unloaded sides of the joint opening, values of relative strain and relative deflection were obtained. These terms appear frequently in the remainder of the report.

In this report, unless otherwise noted, all data pertaining to relative strains and relative

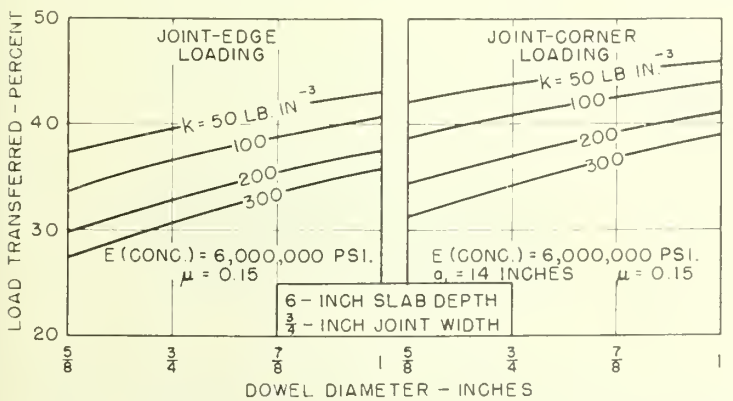

Figure 15.—Effect of dowel diameter on load transfer in the case of a single dowel.

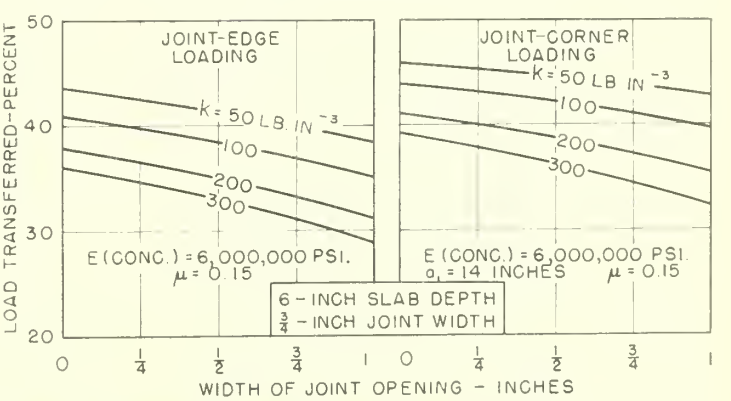

Figure 16.—Effect of width of joint opening on load transfer in the case of a single dowel.

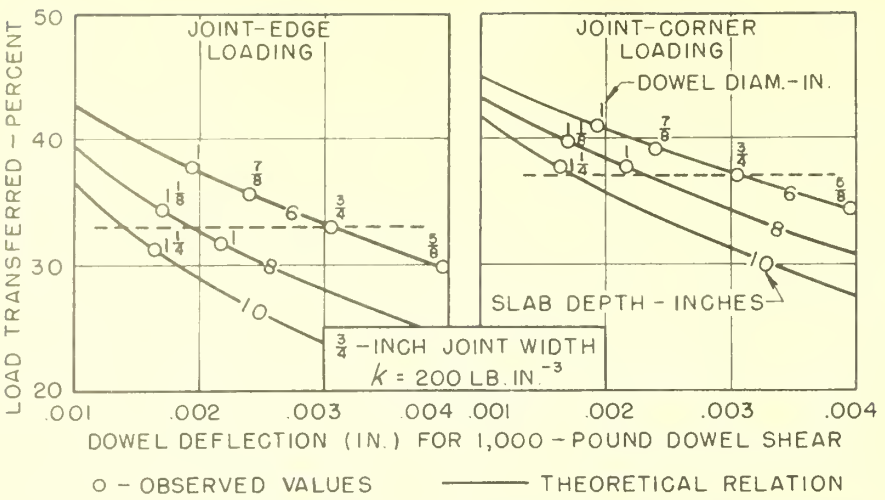

Figure 17.—Effect of dowel deflection (dowel stiffness) and slab depth (pavement stiffness) on load transfer in the case of a single dowel.

deflections are based on the average of two sets of measurements, one set taken with the static test loads applied to one joint edge and the other taken with the same loads applied to the adjacent joint edge.

The data obtained in the testing of the specimen are shown in various ways in figures 5–9, inclusive. These figures illustrate certain characteristics of behavior which were found to be common in all of the subsequent tests.

The relation between the statically applied loads and the corresponding relative deflections of the two slab surfaces, as determined after various numbers of loading cycles, is shown in figure 5. Values of relative deflection were obtained from measurements made with micrometer dials directly above one of the dowels nearest the applied load.

From these data it is apparent that successive increments of load caused progressively smaller increments of relative deflection until the applied load was approximately 5,000 pounds. From 5,000 to 10,000 pounds the load-deflection relation is linear. This indicates that during the application of the first 5,000 pounds of load the dowels were in a state of adjustment in which existing play or looseness was being taken up, and a condition of full bearing was being established. Once this condition had been attained the relation between increments of load and increments of relative deflection became constant. Thus, through the intercept values of the individual slopes on the Y-axis, the graph offers a means for estimating the amount of dowel looseness or play that was present at the beginning of the test, as well as the amount that resulted from repetitive loading.

Causes of dowel looseness

The term dowel looseness as used in this report includes all conditions that tend to prevent the dowel from offering full resistance to load. Conditions which may contribute to dowel looseness are coatings applied to prevent bond, water or air voids in the concrete, particularly under the dowel, shrinkage of the concrete during hardening, and wear of the dowel socket from repeated loading. The magnitude of the initial dowel looseness indicated in figure 5 is 0.0035 inch.

The high-bearing pressures between the dowel and the concrete, particularly in the region above and below the dowel near the face of the joint, tend to break down or wear the concrete during repetitive loading and thus increase whatever looseness may have existed initially. The data in figure 5 indicate that in this test the 2 million cycles of load repetition and stress reversal caused the initial looseness to be increased by an additional 0.003 inch.

The manner in which this looseness increased as the number of load repetitions increased, although evident in figure 5, is shown in more detail in figure 6. In the latter figure the increase in dowel looseness resulting from the repeated application of the 10,000-pound load is traced throughout the 2 million cycles of load application. The individual values were determined by the intercepts on the Y-axis of such curves as are shown in figure 5. It is interesting to note in figure 6 that the increase in looseness, developed during the first 40,000 cycles, equals that developed by the subsequent 1,960,000 cycles.

As stated before, strain values measured in the lower flanges of the two support beams at midspan provide a direct measure of the amount of load being transferred across the joint opening by the load-transfer system. In the case of the representative specimen used in the preliminary studies, figure 7 shows the relation between the statically applied load and the percentage of load transferred after various numbers of application of the 10,000-pound load, as determined from the strain data. In this graph the effect of the gradually increasing dowel looseness, under the repetitive loading, on the percentage of load transferred is quite evident.

This effect is brought out more clearly, however, in figure 8 in which the loss in effectiveness of the load-transfer system is expressed as a percentage of its initial performance and traced throughout the 2 million cycles of load application. In this figure the loss in effectiveness for applied static loads of 10,000, 5,000, and 2,000 pounds is shown. It is apparent that the load-transfer system is much more effective when the slab-end deflection is relatively large as is the case with the larger loads. The relatively rapid increase in dowel looseness during the early part of the test as indicated in figure 6 is reflected in the strain data of figure 8 also.

The data shown in figure 5 indicated that the dowels in the joint of the representative specimen used in the preliminary tests became fully seated under an applied load of approximately 5,000 pounds, and from that load to one of 10,000 pounds the relation between load increments and increments of relative

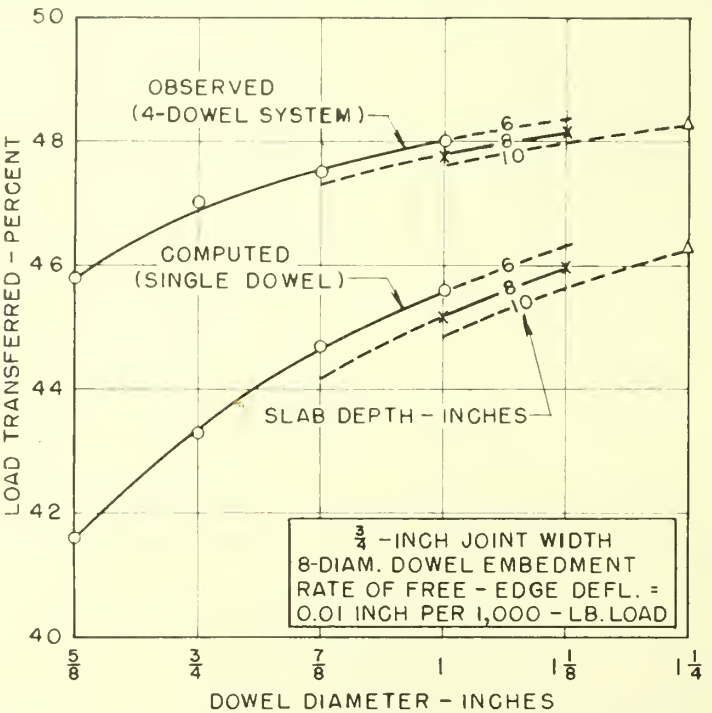
OBSERVED (4-DOWEL SYSTEM)
COMPUTED (SINGLE DOWEL)
SLAB DEPTH - INCHES
¾-INCH JOINT WIDTH
8-DIAM. DOWEL EMBEDMENT
RATE OF FREE-EDGE DEFL. = 0.01 INCH PER 1,000-LB. LOAD
LOAD TRANSFERRED - PERCENT
DOWEL DIAMETER - INCHES

Figure 18 —Observed percentages of load transfer of 4-dowel system compared with computed values of a single dowel for a range in dowel diameters.

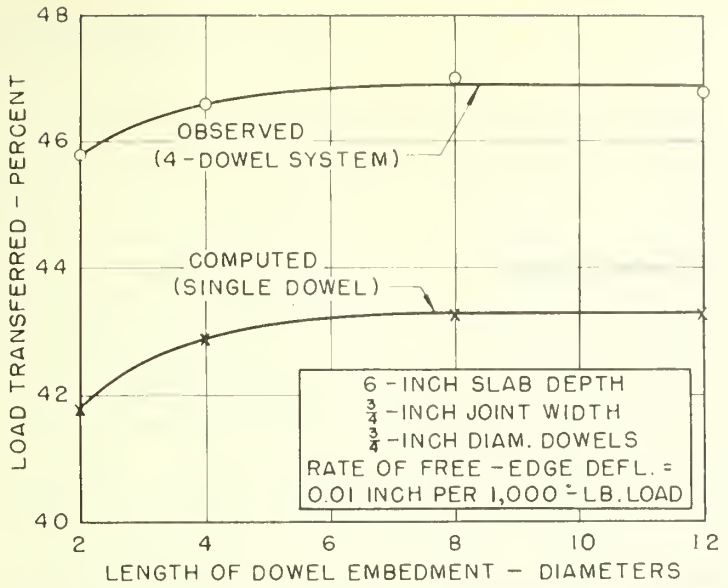

Figure 19.—Observed percentages of load transfer of 4-dowel system compared with computed values of a single dowel for a range in lengths of dowel embedment.

deflection was constant. The same is true for the relation between applied load and load transferred as measured by the strain data. This is brought out in figure 9. It is of interest that once the play and looseness of the system is taken up the effectiveness of the system is relatively high, and even after 2 million repetitions of the application of the 10,000-pound load this effectiveness has remained practically unchanged.

Program of Loading and Observations Adopted

From a study of the data obtained in the preliminary studies, a loading and observation schedule was adopted which was followed in testing all specimens covered by this report, with the exception of a few special cases.

As in the case of the preliminary tests with the representative specimen, the program of repetitive applications of the 10,000-pound (or 15,000-pound) load was interrupted at intervals to permit the application of a series of static test loads for which measurements of relative deflection and strain were made. The static test loads ranged from about 2,000 to 10,000 pounds by 1,000-pound increments. In general, no attempt was made to obtain data under the dynamic load cycle, although in a few tests, oscillograms of the strain data were obtained as will be discussed later. In making the measurements with the various static test loads, the load of a given magnitude was removed before the application of the load of the next higher magnitude.

For each load increment, measurements were made with micrometer dials to determine the relative deflection of the slab surfaces at the joint. With 4 equally spaced dowels in the system, the loading pad was located midway between the 2 central dowels. The measurements of relative deflection were made close to the joint edge and directly over 1 of the 2 central dowels.

Also for each load increment, strains were measured at several points. The relative strains in the lower flanges of the two support beams provided information as to the amount of load being transferred by the dowels. Strain in the upper surface of the concrete specimen in the direction of the joint edge and at midlength of the joint gave information on the transverse bending stress in the concrete. In addition, bending stresses in individual dowels were determined by means of strain gages at static test loads of approximately 2,000, 5,000, and 10,000 pounds.

The measurements just described were made before the beginning of the application of repetitive loads and after 5,000, 15,000, 40,000, 100,000, 200,000, 300,000, 400,000, 500,000, and 600,000 load cycles. This frequency was adequate to establish the behavior pattern of the various specimens without being unduly time consuming.

On the basis of the preliminary studies it was decided to terminate the repetitive loading test on a given specimen after 600,000 cycles. The data showed that significant changes developed before this number of cycles had been applied, and that changes between this number and 2 million cycles were very small. What this represents in terms of traffic is not known, as was stated earlier. However, 600,000 applications of a 10,000-pound load at a given spot on the joint edge of a pavement under traffic must be representative of a considerable period of service on many highways. From the standpoint of the test program, the application of 600,000 cycles of loading required 6 weeks as a minimum, and this seemed to be about all the time that should be devoted to one test specimen.

After preliminary tests and consideration of the data, it was decided to use span lengths for the support beams that would cause a midspan deflection of 0.01 inch for an applied load of 1,000 pounds, assuming no load transfer across the joint. With an applied load of 10,000 pounds, as in a test, the midspan deflection would then be approximately 0.10 inch with no load transfer or 0.05 inch with the assumed ideal condition of complete load

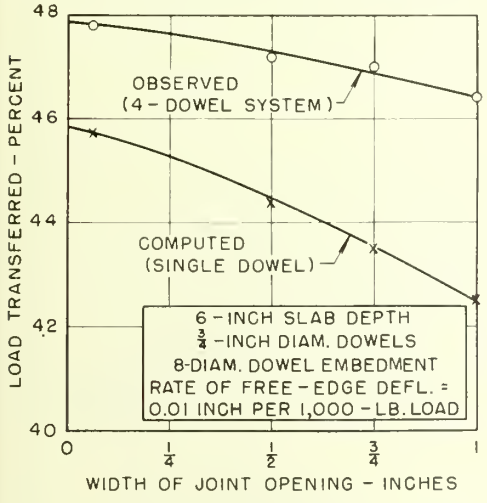

Figure 20.—Observed percentages of load transfer of 4-dowel system compared with computed values of a single dowel for a range in widths of joint opening.

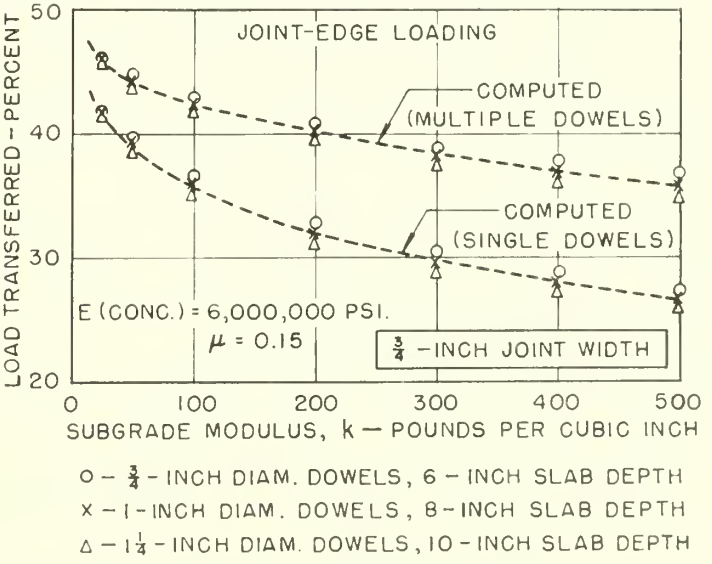
JOINT-EDGE LOADING
LOAD TRANSFERRED - PERCENT
SUBGRADE MODULUS, k - POUNDS PER CUBIC INCH
COMPUTED
(MULTIPLE DOWELS)
COMPUTED
(SINGLE DOWELS)
E (CONC.) = 6,000,000 PSI.
μ = 0.15
¾-INCH JOINT WIDTH
○ - ¾-INCH DIAM. DOWELS, 6-INCH SLAB DEPTH
× - 1-INCH DIAM. DOWELS, 8-INCH SLAB DEPTH
△ - 1¼-INCH DIAM. DOWELS, 10-INCH SLAB DEPTH

Figure 21.—Load-transfer percentages computed for single and multiple dowels for a range in subgrade support values.

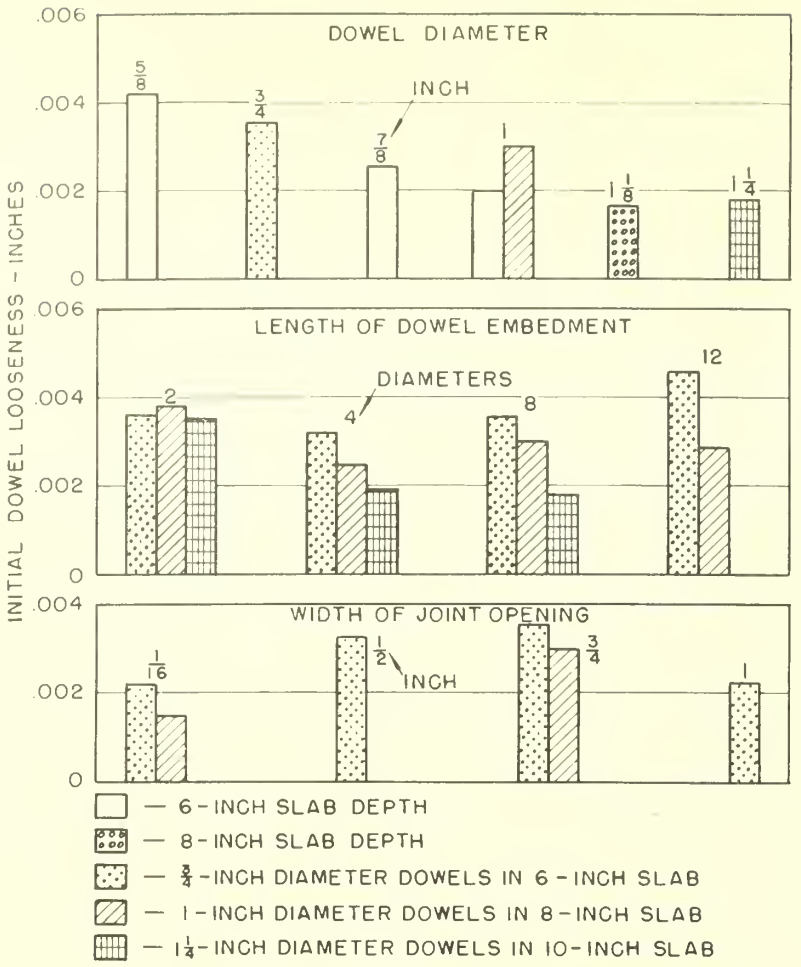

Figure 22.—Data on initial dowel looseness.

transfer. This deflection rate is somewhat greater than that which usually prevails at a transverse joint edge of a fully supported concrete pavement slab. Many slab ends are not fully supported, however, and deflection rates of the magnitude just mentioned have been measured. From the testing standpoint it was believed that data developed at the rate selected would be more sensitive to structural deterioration in the load-transfer system than those which were developed at a lesser rate.

Twenty-nine specimens were tested in the studies of the three variables of dowel diameter, length of dowel embedment, and width of joint opening. One specimen (the first) was carried through only 57,000 loading cycles and the data are not included; two were used in special tests outside the program. Thus, 32 specimens in all were constructed.

Of the 29 specimens in the studies of the 3 major variables, 8 were used in studies of the effects of dowel diameter, 20 in the studies of length of dowel embedment, and 8 in the studies of the effect of width of joint opening. The data from certain specimens could be used for more than one comparison which accounts for the apparent discrepancy in numbers of specimens.

The dowel diameters used in the tests were 5/8 inch, 3/4 inch, 7/8 inch, 1 inch, 1 1/8 inches, and 1 1/4 inches. In the studies of the effect of length of dowel embedment, a constant width of joint was used throughout; and the lengths of embedment, expressed in dowel diameters, ranged from 2 to 12. The actual dowel lengths are shown in table 1. The widths of joint opening used in the test were 1/16 inch, 1/2 inch, 3/4 inch, and 1 inch.

The amount of load that can be transferred by a dowel or dowel system depends upon (1) the load carrying capacity of the dowel under the most favorable conditions, (2) the amount by which this optimum capacity is reduced by what has been termed initial dowel looseness in this report, and (3) the amount by which initial capacity to transfer load has been reduced by subsequent repetitive loading. Accordingly, the data obtained in the major part of the test program being reported are presented and discussed in the order mentioned.

Table 1.—Actual dowel lengths included in the study of length of dowel embedment

Diameters of dowels (inches)	Lengths of dowels—in inches			
3/4	3.75	6.75	12.75	18.75
1	4.75	8.75	16.75	24.75
1 1/4	5.75	10.75	20.75	-----

Load Transfer Under Ideal Conditions

In the discussion of figure 5 earlier in the report, it was noted that the relation between increments of applied load and increments of relative deflection of the abutting slab edges did not become constant until a static test load value of about 5,000 pounds was reached. It was concluded that at this point a condition of full bearing of the dowel in the concrete socket had been established. The linear portion of the relation which was found for the higher loadings thus may be taken as representative of true elastic deformation of the dowel and the concrete, and hence a means by which the capacity of the system to transfer load under the most favorable conditions can be determined.

It was decided that for studying the effects of varying dowel diameter, length of dowel embedment, and width of joint opening a useful index would be the amount of dowel deflection resulting from a shear load of 1,000 pounds—the term dowel deflection representing relative deflection with the dowel in full bearing on the concrete. Such an index could be obtained from the linear portion of the load-relative deflection relations, such as

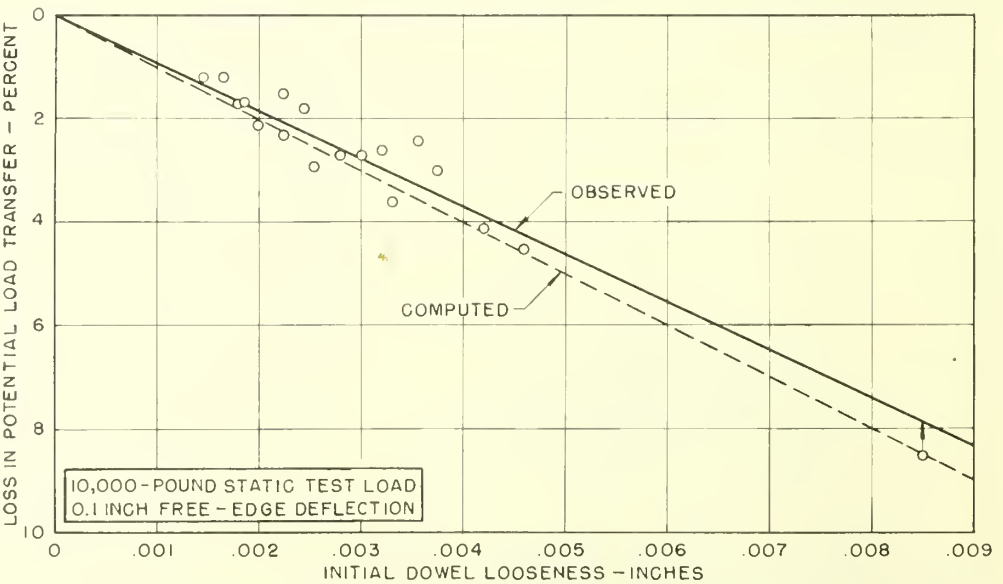

Figure 23.—Relation between initial dowel looseness and loss in potential ability to transfer load.

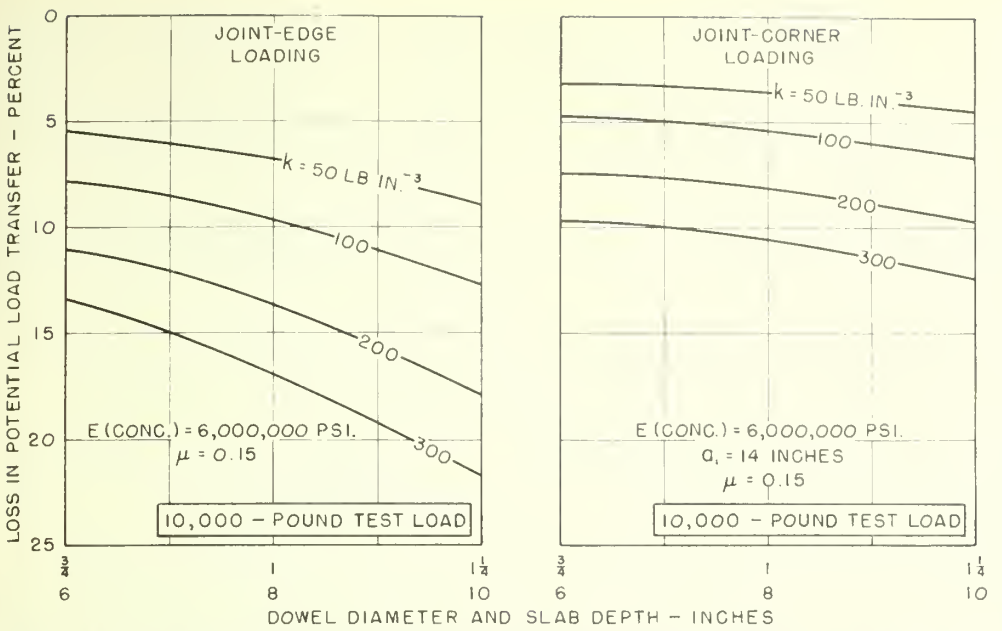

Figure 24.—Effect of initial dowel looseness on loss in potential ability to transfer load for certain combinations of dowel diameter and slab depth.

those shown in figure 5, by any one of the three methods described briefly in the following paragraphs.

Method 1.—Loads ranging from 5,000 to 10,000 pounds by 1,000-pound increments were applied. Relative deflections were measured over each of the four dowels. Average shear in the load-transfer system was determined from strain values in support beams and related to average relative deflection values. This method was used in 19 tests.

Method 2.—Applied loads were the same as in method 1. Relative deflections were measured over one dowel adjacent to the load only. Shear value for this particular dowel was arrived at by distributing total shear among the four dowels according to value of bending strains in each one. Each point defining the relation is an average of 10 test values. This method was used in 18 tests.

Method 3.—After completion of the regularly scheduled test on a 4-dowel system, the 2 outer dowels were cut through so that only the 2 central dowels remained active. Average shear and relative deflection values were obtained as in method 1. This procedure was used in 12 tests.

In four tests all three methods were used and in the majority of the remainder two methods were used. Typical data from the three methods on a single specimen are shown in figure 10. For convenience in presentation the straight lines were drawn to pass through the origin (although the origin for method 3 is off the graph). It was concluded that essentially the same index value was obtained by each method. Where two or three methods were used on a single specimen, the values obtained were averaged.

Effect of Design Features on Load Transfer

In figures 11–14 the dowel-deflection index values just described have been utilized to study the effects of varying the principal design features of dowel diameter, length of dowel embedment, and width of joint opening. The index values provide a measure of the relative load-transfer capacity for the particular conditions involved. The larger the index value, the less effective is the system.

Dowel diameter

In figure 11, the dowel-deflection index data are utilized to show the effect of variations in dowel diameter. These data are from tests in which the width of joint opening was ¾ inch. As indicated, four of the values were for a 6-inch slab depth, two for an 8-inch slab depth, and one for a 10-inch slab depth. Thus some information is provided on the influence on dowel deflection of the depth of concrete above and below the dowel.

If one considers a dowel as a cantilever being deflected by a vertical force equal to the shear on the dowel, it is apparent that the deflection at the point of load application will be the result of (1) the deformation of the concrete under the bearing load exerted by the dowel, (2) the angular change in direction of the dowel axis resulting from the deformation of the concrete, and (3) the elastic bending of the dowel itself.

In his analysis of dowel design, Friberg (*4*) considered each of these factors and combined them in a single formula for the deflection of a dowel crossing a joint. It is of interest to compare the values of dowel deflection observed in the experiments being reported with corresponding values computed by Friberg's formula (see appendix), utilizing the elastic properties which existed in the present tests. This comparison is made in figure 12 in which the deflection values determined experimentally for the four dowel sizes in the specimens of 6-inch depth are shown as plotted points, while the relation obtained with the formula is shown as the solid line. It is apparent that the relation between dowel diameter and dowel deflection is an exponential one in each case, although the value of the exponent found in these experiments is somewhat different than that in the theoretical formula. For the dowel diameters greater than ¾ inch, it is apparent also that the deflection values found in the present experiments are quantitatively not greatly different from those computed by the formula.

Length of dowel embedment

The deflection index data may be used also to study the effect of varying the length of embedment (or bearing on the concrete) on the ability of the dowel to resist loads. In figure 13, dowel-deflection index values are shown for 3 dowel diameters (¾ inch, 1 inch, and 1¼ inches) and for a range of lengths of embedment from 2 to 12 diameters. The width of joint opening was ¾ inch in all cases. It will be observed that the greater the stiffness of the dowel itself, the less the length of its embedment, in terms of dowel diameter, influences the deflection.

The relation shown for the ¾-inch diameter

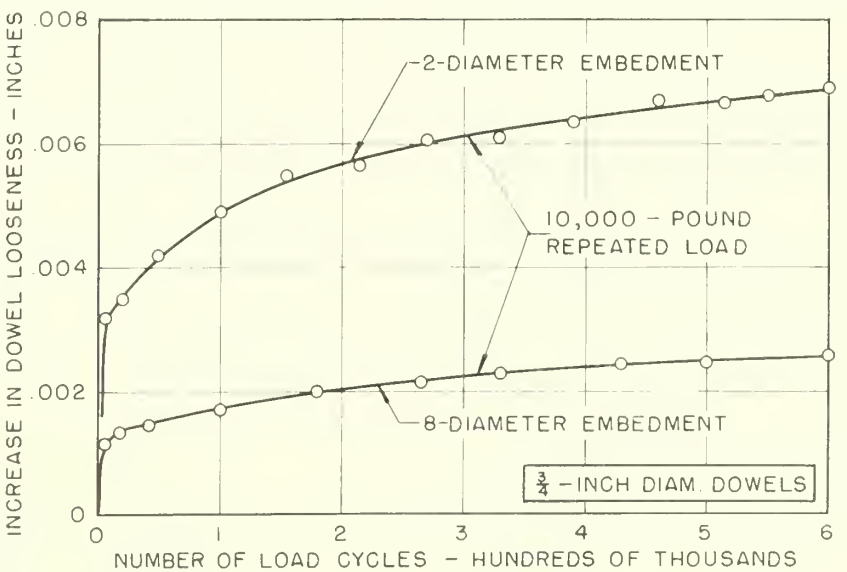

Figure 25.—Effect of length of dowel embedment on the development of dowel looseness under repetitive loading.

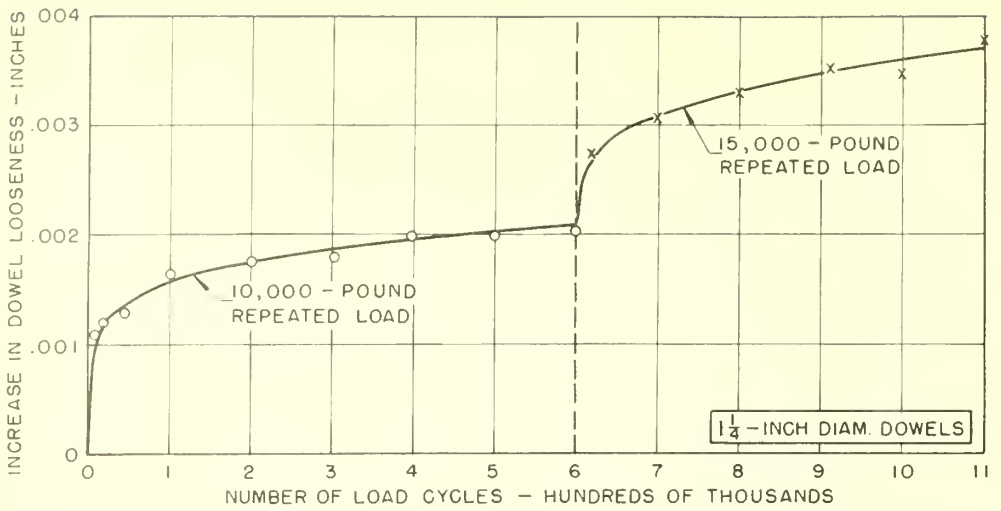

Figure 26.—Effect of increasing the magnitude of the repeated load on the development of dowel looseness.

dowel indicates that for a width of joint opening of ¾ inch an embedment of about 8 diameters is required to develop maximum resistance. For a 1-inch diameter dowel and a ¾-inch joint opening, maximum resistance is obtained with an embedment of about 6 diameters, whereas for a 1¼-inch diameter dowel an embedment of about 4 diameters develops maximum resistance. It should be borne in mind that these embedment lengths are for concrete of relatively high strength and for dowels that are embedded with unusual care. Whether the same relations would hold for concrete of lesser strength can only be determined by further tests. Such tests should be made, as the present data point to the possibility of a considerable savings in steel requirements for dowels, particularly with the larger diameter dowels now coming into use.

Width of joint opening

In figure 14 (p. 7) the dowel-deflection index values are arranged to show the effect of increasing the width of joint opening. The data are for a dowel embedment of 8 diameters in all cases. The dowel diameters are ¾ inch, 1 inch, and 1¼ inches and the depth of the slab varies with dowel diameter in the manner indicated. As would be expected, there is a marked increase in dowel deflection as the width of joint opening is increased, the deflection of a given dowel size being approximately doubled as the width of opening is increased from 1/16 to 1 inch. This indicates that a given dowel size provides appreciably more load transfer in a contraction joint than in an expansion joint, other conditions being the same. The influences of dowel diameter and the depth of the concrete above and below the dowel that were noted earlier are apparent in this graph also.

Significance of Dowel Deflections

The effectiveness of a load-transfer system can be measured by the amount it reduces free-edge load stress or by the amount it reduces free-edge load deflection. The two criteria are related but are not necessarily the same, and their relative importance depends upon the overall structural design of the pavement. In the tests being reported, the effects of repetitive loading on slab deflections rather than on slab stresses are being studied.

One measure of the extent of load being transferred across a pavement joint by a dowel or other load-transfer unit is the amount by which the magnitude of the free-edge deflection of the pavement, under a given applied load, is reduced by the presence of the load-transfer unit. This relation was discussed in the report of the Arlington tests (*2*) (see PUBLIC ROADS, vol. 17, No. 7). Subsequently it was expressed for the case of a single dowel in a formula by Friberg in his ASCE paper (*4*) and by Richart and Bradbury in discussions of Friberg's Highway Research Board paper (*4*). While the form of the expression used by the three authors differed somewhat, the relation expressed was basically the same. The derivation is based on the premise that the deflection of the pavement edge on which the load is applied must equal the deflection of the dowel plus the deflection of the adjacent slab edge. Thus the load applied to the latter through load transfer is equal to the dowel shear. For the relation to hold, it is implicit that either there be no looseness of the dowel in its socket or any existing looseness be eliminated.

The proportionate part of the applied load that is transferred to the adjacent slab by the dowel may be obtained from the following expression:

$$p = \frac{1}{2 + \frac{y_d}{y_p}} 100 \qquad \text{formula (1)}$$

In which:

p = Proportion of load transferred, in percent.

y_d = Dowel deflection caused by unit shear, in inches.

y_p = Free-edge deflection of the pavement caused by unit load, in inches.

The expression indicates that under the ideal conditions assumed and within the elastic range of the materials involved the percentage of load transferred is independent of the load magnitude. It shows also that the percentage depends upon the relative stiffnesses of the dowel and of the pavement and, in turn, those factors which affect the stiffness of either.

In figures 15–17, experimental dowel stiffness (deflection) data taken from figures 11 and 14 have been combined with pavement stiffness (deflection) values computed with Westergaard's equations (see appendix) in the expression given as formula 1 to bring out the inter-relationships that exist between dowel size, width of joint opening, slab depth, and modulus of subgrade reaction. These comparisons apply to a single dowel.

Figure 15 (p. 7) shows, for the two cases of joint-edge and corner loading, the relations between dowel diameter and percentage of load transferred, as the modulus of subgrade reaction k is varied from 50 to 300 pounds per cubic inch (p. c. i.). It is indicated for the con-

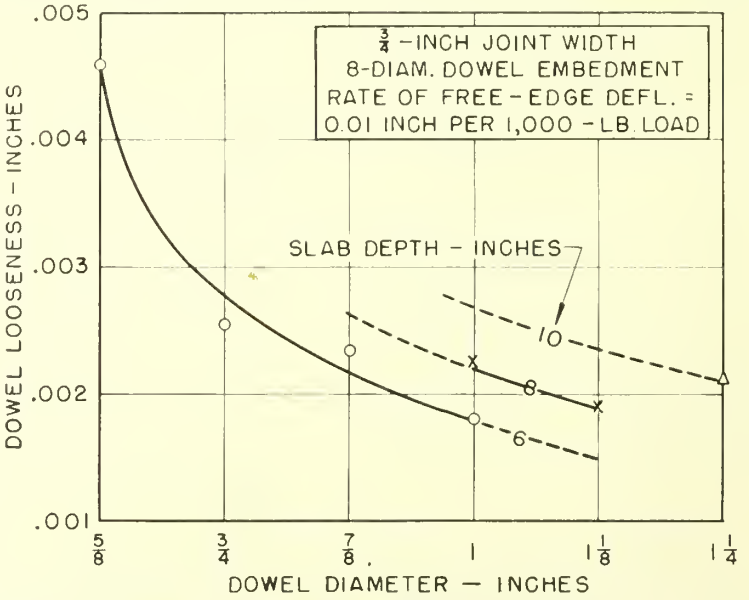

Figure 27.—Relations between dowel diameter and dowel looseness resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

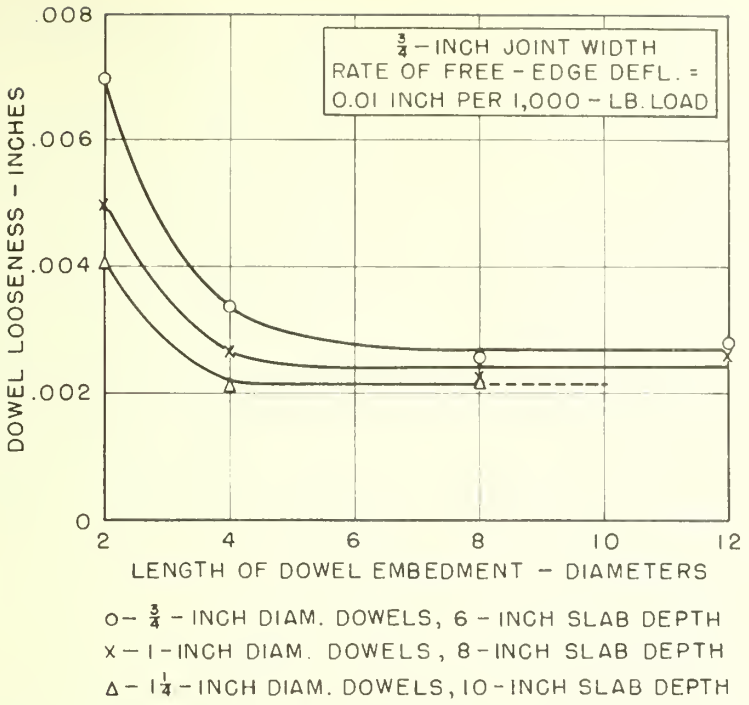

Figure 28.—Relations between length of dowel embedment and dowel looseness resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

ditions stated that an appreciable increase in the percentage of load transferred is obtained by increasing the diameter of the dowel, the rate of the increase becoming greater as the supporting power of the subgrade becomes greater. For example, as the subgrade modulus, k, is changed from 50 to 300 p. c. i., the rate, as expressed by the increase in percentage of load transferred per ⅛-inch increase in dowel diameter, increases from 1.90 to 2.77 for edge loading and from 1.23 to 2.57 for corner loading.

Two important structural benefits are obtained as the diameter of the dowel is increased: increased dowel rigidity with better load-transfer ability and greater bearing area on the concrete with reduced bearing pressures immediately above and below the dowel.

The effect of changing the width of the joint opening on the percentage of load transferred by a single dowel is shown in figure 16 (p. 7) for the two cases of loading with the same pavement depth and range of values of the subgrade modulus, k, mentioned in the discussion of figure 15. It is indicated by these relations that as the stiffness of the subgrade becomes greater the effect of joint width on load transfer becomes greater also.

In figure 17, the theoretical relation between the percentage of load transferred and the dowel stiffness is shown in a somewhat different manner. By means of the Westergaard equations and for the conditions stated, the relation between load and deflection was established for the free edge and free corner of pavement slabs of 6-, 8-, and 10-inch depths. These deflection values were used in formula 1 to obtain the percentage of load transferred by a single dowel as the stiffness of the dowel was varied for each of the three slab depths just mentioned. The resulting relations are shown in figure 17 as a family of three full-line curves. In this group of theoretical relations, the observed experimental dowel deflection values have been shown as plotted points.

Figure 17 is of particular interest because any horizontal line, such as the dashed-line shown, indicates a relation between slab depth and the size of dowel necessary to accomplish a given percentage of load transfer. For example, it indicates that for pavements of 6-, 8-, and 10-inch depths, dowels of ¾-, 1 1/16-, and 1⅜-inch diameters, respectively, might be expected to effect the same percentage of load transfer for the joint-edge loading; and that dowels of ¾-, 1-, and 1¼-inch diameters would do the same for the joint-corner loading.

It is realized that the experimental data on which this relation is based are somewhat meager, particularly for the 8- and 10-inch slab depths. However, it is believed that the analysis is a valid one, and it is to be noted that the relation indicated is concordant with the recommendations of the American Concrete Institute (*8*). It might be stated as an approximate rule, as follows: For round steel dowels at 12-inch spacing with a joint opening of ¾ inch or less, the diameter of the dowel in eighths of an inch should equal the pavement depth in inches.

Load-Transfer Capacities of Single and Multiple Dowels Compared

The discussion of load transfer up to this point has been confined to that provided by a single dowel functioning under ideal conditions. In the investigation being reported for reasons stated earlier, a system of four dowels was used in each specimen and the deflections of the joint edge under the loads used were relatively large as pavement deflections go. The tests do provide some comparative data, however, and it is of interest to examine the comparison between the single dowel and the 4-dowel system for the three major variables of dowel diameter, length of dowel embedment, and width of joint opening as shown in figures 18, 19, and 20, respectively.

In preparing these figures the load-transfer capacity values for the 4-dowel system were obtained in the manner described in connection with figure 9, each value being based on 10 individual tests with a given dowel system. The comparative values for the single dowel were computed by formula 1 using dowel stiffness data given in figures 11, 13, and 14, and a free joint-edge deflection rate of 0.01 inch per 1,000 pounds of applied load. It is important to keep in mind that the relations shown in these graphs represent performance under the most favorable or optimum conditions. This and the relatively high deflection rate which applies to these test values explain the high percentages of load transfer shown in the graphs.

Stated in another way, the percentages shown would be found only where the dowel functioning was perfect and where there was relatively weak subgrade support on both sides of the joint. Had the deflection rate been smaller, as would be the case with a stronger subgrade, or had the dowel action been less than perfect, or both, the general level of load transfer would have been lower. Regardless of the magnitude of the load-transfer percentages, the comparison between the performance of the 4-dowel system as observed in the tests and that of the single dowel as computed by the formula is believed to be valid and of interest in the analysis of the data.

As will be observed in figures 18–20, the trends for the three major variables of dowel diameter, length of dowel embedment, and width of joint opening are essentially the same for the 4-dowel system and the single dowel. It is evident that a good correlation exists throughout; and, for a given percentage of load transferred by the single dowel, an essentially constant numerical difference exists between the load-transfer capacity of the 4-

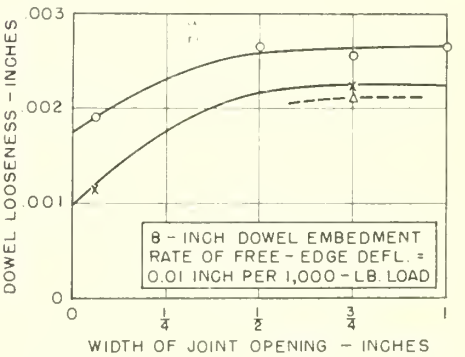

Figure 29.—Relations between width of joint opening and dowel looseness resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

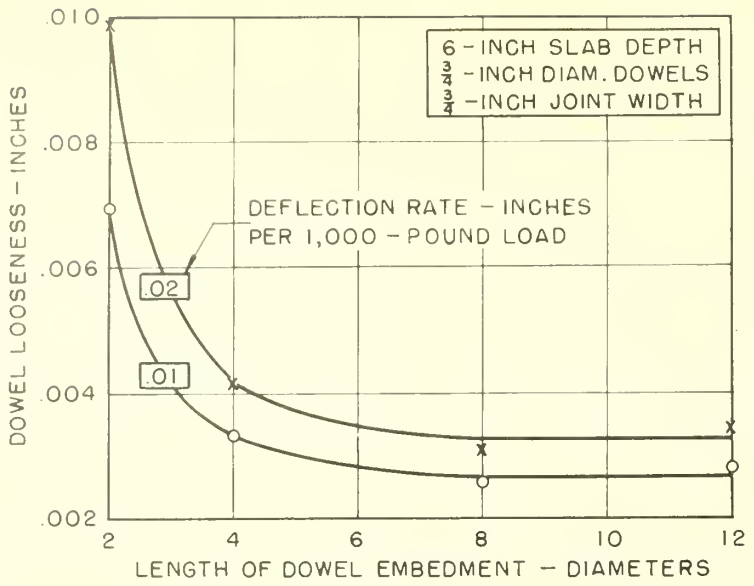

Figure 30.—Comparison of dowel looseness developed at 0.01- and 0.02-inch free joint-edge deflection rates after 600,000 cycles of a 10,000-pound load.

dowel system and the single dowel, irrespective of relations for dowel diameter, length of embedment, or width of joint opening.

Whether or not this generalization would hold for other rates of joint-edge deflection was not established definitely by these tests. However, an analysis of the data from six tests in which the free-edge deflection rate was double that shown in figures 18–20 indicates that it might apply; and, on the assumption that it does, figure 21 (p. 9) was prepared to show the general effect of the stiffness of the support afforded by the subgrade on the load-transfer capacity of a dowel or dowel system.

The computed values shown as plotted points were obtained with formula 1, utilizing dowel deflection rates from the tests being reported, slab-edge deflection rates computed by means of the Westergaard equations for the several values of the subgrade modulus, k, and the relation between single and multiple dowel load-transfer characteristics established by the data shown in the preceding graphs. It will be noted that the observed dowel deflection rates for the ¾-inch diameter dowel in a 6-inch depth slab, the 1-inch diameter dowel in an 8-inch depth slab, and the 1¼-inch diameter dowel in a 10-inch depth slab were utilized in the computations.

Of the several comparisons available for ideal conditions represented in figure 21, probably the most significant is the one which shows how the stiffness of the subgrade support influences the load transfer that can be obtained with a given system. As has been stated, it was for this reason that figure 21 was prepared.

Effect of Initial Dowel Looseness on Load Transfer

The discussion thus far has been concerned principally with dowel performance with looseness eliminated. Early in the report, dowel looseness was defined as including all conditions that tend to prevent the dowel from offering full resistance to load. Among the conditions which contribute to dowel looseness were mentioned coatings used to prevent bond, air or water voids in the concrete, shrinkage of the concrete during hardening, and wear of the dowel socket during repetitive loading.

It may safely be assumed that some initial looseness will always exist in doweled joints as they are built in practice, and the effect of such looseness on load transfer is a matter of considerable interest.

It is obvious that a dowel or dowel system does not begin to function at maximum efficiency until all initial looseness is taken up by deflecting the pavement on the loaded side of the joint farther than would be necessary if initial looseness were not present. Thus the effect of looseness is to reduce the potential usefulness of the load-transfer system by an amount that depends upon the degree of dowel looseness that exists.

If it is assumed that slab deflection is proportional to applied load and, in general, this is a valid assumption (see PUBLIC ROADS, vol. 23, No. 8) (*2*), the loss in potential load-transfer capacity resulting from initial looseness in the load-transfer system can be expressed for a given load as follows:

$$\Delta p = \frac{l_i}{y} 100 \qquad \text{formula (2)}$$

In which:

Δp = Loss in potential load-transfer capacity, in percent.

l_i = Initial looseness in load-transfer system, in inches.

y = Free-edge deflection of the pavement caused by the load in question, in inches.

The expression indicates that the loss in potential load carrying capacity, while independent of the capacity of the system, increases directly with the magnitude of the

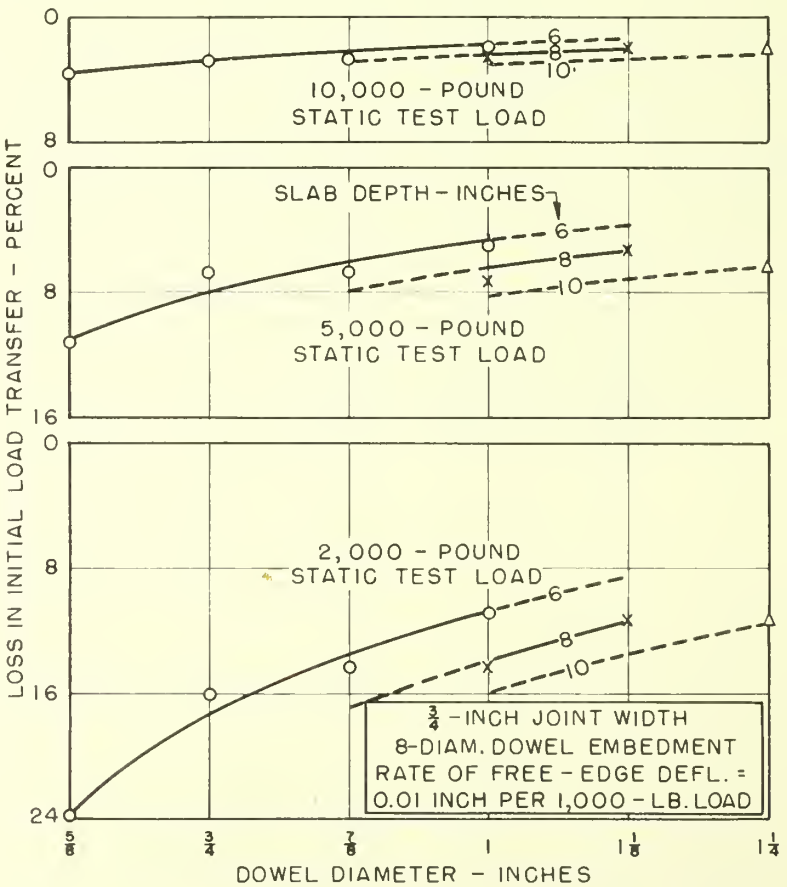

Figure 31.—Relations between dowel diameter and loss in initial capacity to transfer load resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

initial looseness and decreases as the magnitude of the free-edge deflection increases.

In figure 22 are shown data on the initial looseness for the dowel systems of various specimens tested in the investigation being reported. The values were determined in the manner shown in figure 5, that is, by extending the linear part of the relation shown before repetitive loading was started to its intercept with the Y-axis. In figure 22 the values are grouped according to the design variables of dowel diameter, length of dowel embedment, and width of joint opening.

It is apparent that the values shown tend to be erratic among the various specimens. The magnitudes are all relatively small, ranging from 0.0015 to 0.0045 inch. There is no apparent trend with either length of dowel embedment or with width of joint opening. There does appear to be a rather systematic decrease in initial looseness as the diameter of the dowels is increased, however.

The effect of initial looseness of a dowel system in reducing its potential capacity to transfer load is shown for stated conditions in figure 23. The loss values in this figure are differences between load transfer for the ideal condition of no initial looseness and that observed for the first load cycle when only initial looseness was present. The theoretical or computed relation was established with formula 2, using a free-edge pavement deflection of 0.1 inch and assumed values of initial dowel looseness. Figure 23 shows good agreement of observed data with the relation as computed.

The plotted point on the right-hand side of figure 23 at an initial looseness value of 0.0085 inch is of interest. It represents a condition encountered in one of the first specimens to be constructed. The dowels were held in a partition form of dry wood. The swelling of the wood as it absorbed moisture from the concrete is believed to have caused enough vertical movement of the dowels to develop the unusual looseness observed in this specimen. Subsequently, the wood partition was submerged in water for at least 24 hours, and this resulted in a marked improvement. However, after a few specimens had been made, the use of wood was abandoned and a removable steel partition was built and used in the construction of a majority of the specimens. This eliminated the possibility of any water absorption.

Figure 24 shows certain derived relations, based on observed data and theory, designed to bring out the practical significance of initial looseness in a doweling system. The test specimens selected for analysis were as follows:

Dowel diameter, in inches	*Slab depth, in inches*	*Initial looseness, average all specimens, in inches*
¾	6	0.0032
1	8	.0026
1¼	10	.0024

The relations shown in figure 24 were obtained for these three cases by means of formula 2, utilizing the Westergaard formulas for computing values of free-edge deflection for a 10,000-pound load and selected values of the modulus of subgrade reaction. These relations are useful in making evident the important losses in load-transfer capacity that can be caused by relatively small degrees of initial looseness in the doweling system, particularly for firmer subgrades and thicker pavements where load-deflection magnitudes are small.

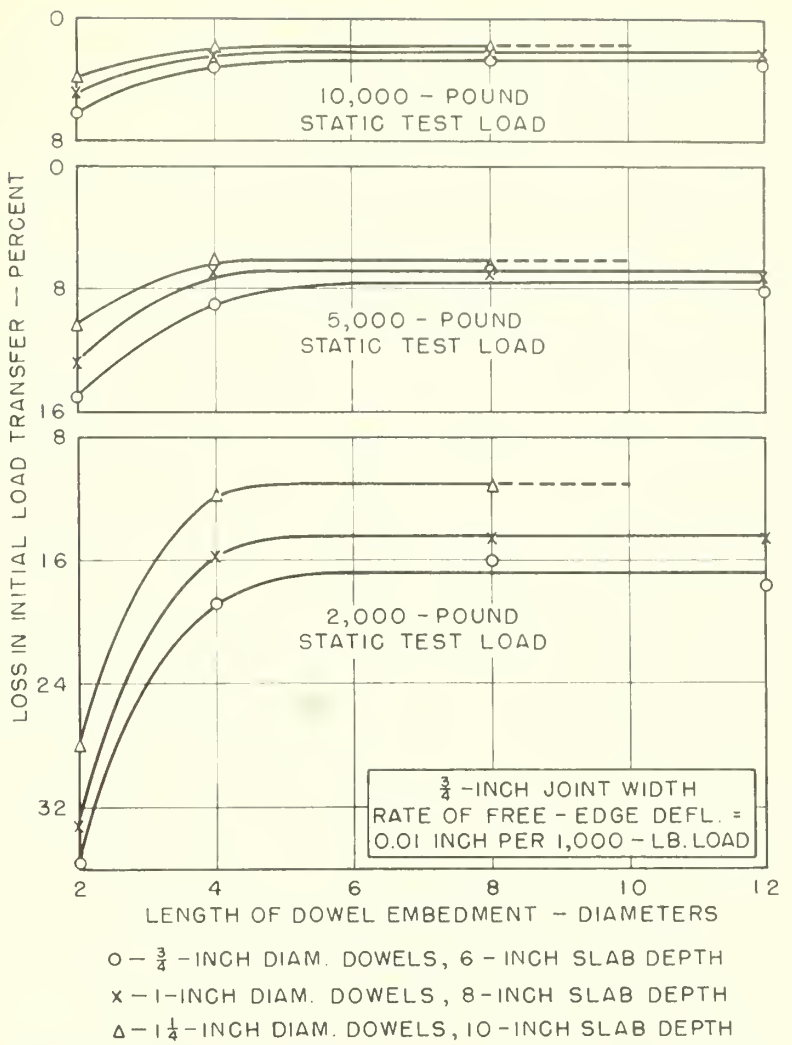

Figure 32.—Relations between length of dowel embedment and loss in initial capacity to transfer load resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

Effect of Repetitive Loading on Load Transfer

The load repetitions and load reversals, as applied in these tests, caused a progressive increase in the looseness of the dowels as it existed initially. The increase in looseness was accompanied by a progressive loss in load-transfer capacity, as would be expected. This change under repetitive loading is attributed to a breakdown or wear in the concrete of the dowel socket above and below the dowel, particularly in the region near the joint face, caused by the repeated application of intense bearing pressure.

It is of interest to note that after as many as 2 million cycles of the application of a 10,000-pound load at a joint containing four ¾-inch diameter dowels, the wear or "funneling", as it is sometimes called, was so small as to be undetectable by visual examination. The change was readily measurable with the instrumentation used in the tests, however. Typical test data are shown in figures 25 and 26.

In figure 25, the effect of embedment length is shown to be an important variable affecting both the magnitude and the rate of development of the looseness caused by repetitive loading.

The load magnitude is shown in figure 26 to be an important variable. After the rate of increase of looseness had reached a very small value following the application of 600,000 cycles with the 10,000-pound load, an increase of load to 15,000 pounds brought about an immediate increase in the rate of development of further looseness, a change which had not completely stabilized after an additional 500,000 cycles of this loading.

These figures are of interest because they show the manner in which looseness of the dowel develops under repetitive loading and indicate some of the variables involved.

In these tests, measurements of the relative deflection of the joint edges provide information concerning the initial looseness in the load-transfer system and the manner in which this initial looseness increases as the dowels are subjected to repetitive loading. Measurements of relative strain provide information on the changes in the load transferring ability of the dowel system as the repetitive loading proceeds. From the data obtained, it is pos-

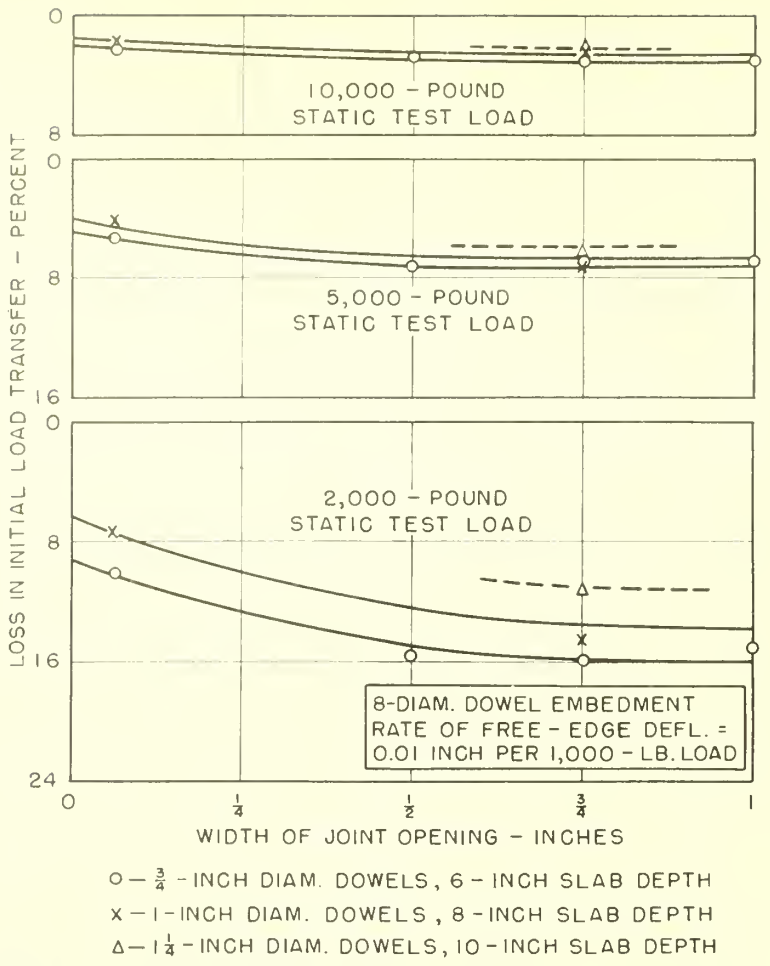
10,000 - POUND
STATIC TEST LOAD
5,000 - POUND
STATIC TEST LOAD
2,000 - POUND
STATIC TEST LOAD
8-DIAM. DOWEL EMBEDMENT
RATE OF FREE - EDGE DEFL. =
0.01 INCH PER 1,000 - LB. LOAD
LOSS IN INITIAL LOAD TRANSFER - PERCENT
WIDTH OF JOINT OPENING - INCHES
O — ¾ - INCH DIAM. DOWELS, 6 - INCH SLAB DEPTH
X — 1 - INCH DIAM. DOWELS, 8 - INCH SLAB DEPTH
Δ — 1¼ - INCH DIAM. DOWELS, 10 - INCH SLAB DEPTH

Figure 33.—Relations between width of joint opening and loss in initial capacity to transfer load resulting from 600,000 cycles of a 10,000-pound load (base measurements obtained at first cycle).

sible to relate the data from the two types of measurement.

Considering first the changing magnitude of dowel looseness as the dowel system is subjected to load repetition and complete stress reversal, the data have been examined from the standpoint of the three design variables—dowel diameter, length of dowel embedment, and width of joint opening. The effects of each of these variables on the development of dowel looseness under repetitive loading are shown in figures 27, 28, and 29, respectively. In each of these graphs, the magnitude of the looseness resulting from the application of 600,000 cycles of a 10,000-pound load forms the basis of comparison and is related to the several design variables for the other test conditions stated.

It is apparent from the relations shown in these graphs that the dowel looseness caused by the application of 600,000 cycles of repetitive loading is, in general, rather small—its magnitude being no greater than that which existed before the application of the loads, as shown in figure 22.

The values of dowel looseness shown in figures 27–29 were obtained in tests with a free-edge deflection rate of 0.01 inch per 1,000 pounds of load. A limited amount of comparable data were obtained in tests in which a free-edge deflection rate of 0.02 inch was used. The comparison shown in figure 30 relates length of dowel embedment to the dowel looseness, developed by 600,000 cycles of repetitive loading, for each of the free-edge deflection rates mentioned. It is indicated that the deflection rate has little effect on the relation between length of dowel embedment and dowel looseness. The actual magnitude of the looseness is affected, however, since the greater the free-edge deflection rate the greater the looseness, other conditions being constant.

The rate of development of dowel looseness under repetitive loading would be expected to depend primarily upon three factors: intensity of the bearing pressure between the dowel and its concrete encasement, the number of load applications, and the strength of the concrete. The present tests have thrown considerable light on the effects of both pressure intensity and the number of load applications, but not on the effects of variations in the strength of the concrete because, as was stated earlier, every effort was made to have the concrete uniform and of high quality.

Since the development of dowel looseness under repetitive loading leads directly to a reduction in the load transferring ability of a dowel system, it is of interest to examine the data on loss in load transfer from such loading as determined from the strain measurements.

Figures 31–33 show the extent to which the initial load-transfer capacity of typical load-transfer systems deteriorated under 600,000 cycles of repetitive loading in which a 10,000-pound load was applied alternately on either side of the joint in each cycle.

The data in these graphs, expressed as percentages of the initial capacity of the particular joint system to transfer load, were taken from relations such as those in figure 8. They have been arranged in the three graphs to show how the three design variables of dowel diameter, length of dowel embedment, and width of joint opening affect the loss in load-transfer capacity caused by repetitive loading. In each case, observed values are for static test loads of 2,000, 5,000, and 10,000 pounds. The effect of any structural deterioration that develops is always most readily apparent under the 2,000-pound static test load.

Referring to figure 31, the data indicate that as the diameter and corresponding bear-

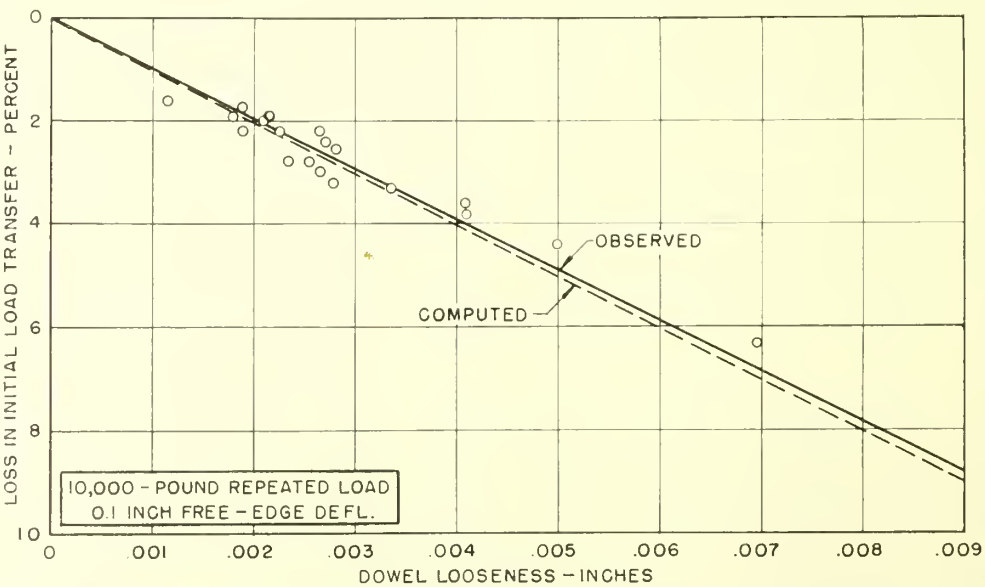
OBSERVED
COMPUTED
10,000 - POUND REPEATED LOAD
0.1 INCH FREE - EDGE DEFL.
LOSS IN INITIAL LOAD TRANSFER - PERCENT
DOWEL LOOSENESS - INCHES

Figure 34.—Relation between dowel looseness and loss in initial capacity to transfer load, after 600,000 cycles of a 10,000-pound load.

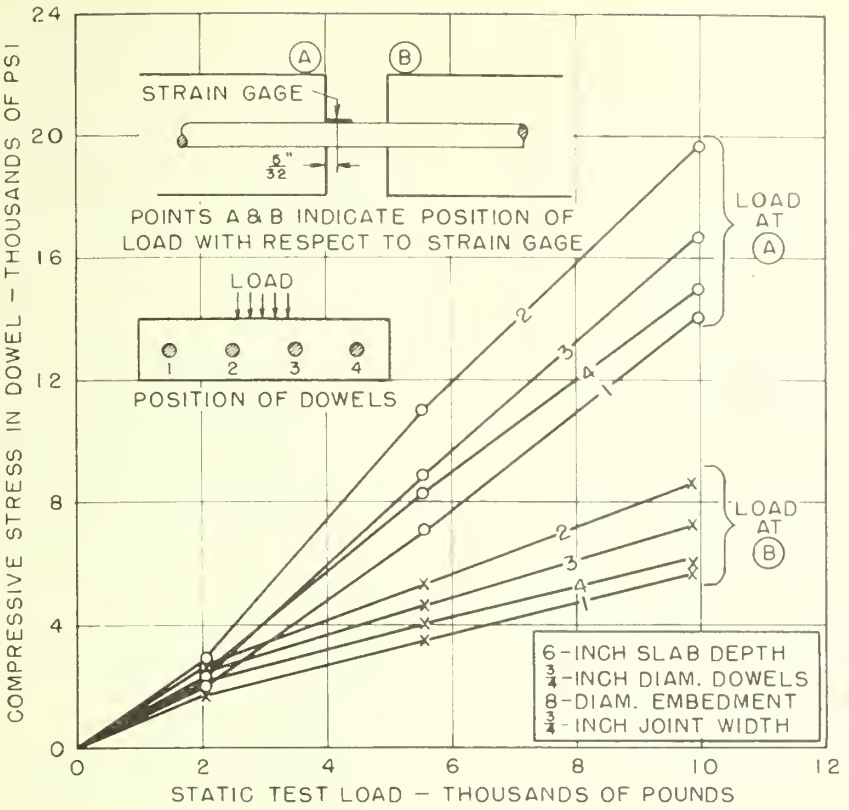

Figure 35.—Example of relations between load applied at indicated joint edges and stress in dowels at indicated gage point before the application of repetitive loading.

ing area of the dowel is increased, there is a marked improvement in the degree to which the initial capacity to transfer load is preserved during repetitive loading. The important influence of unit pressure in the dowel seat is emphasized by the data relating to dowel diameter.

In figure 32 the effects of varying the length of dowel embedment are shown. Although the point at which curvature begins in these relations is not precisely established by the data, it is indicated that where the embedment length is 8-dowel diameters or more there is no effect on the loss of initial load-transfer capacity. Where the embedment length is 4-dowel diameters, there is little effect for the 1-inch and 1¼-inch diameter dowels; while for an embedment length of 2-dowel diameters, there is a marked effect with all 3 dowel diameters. Thus, so far as losses of load-transfer capacity under repetitive loading are concerned, it is indicated that other design considerations will probably determine the length of dowel embedment.

In figure 33, the data are arranged to show the effect of varying the width of joint opening on the losses of the initial load-transfer capacity which were caused by the 600,000 cycles of repetitive loading. It is indicated that as the width of opening is increased from 1/16 to 1 inch there is an increase in the magnitude of the loss. It is indicated also that the rate diminishes rapidly with increase in width of joint opening. The data show one of the reasons why repeated loading would tend to cause less impairment of initial load-transfer efficiency in a contraction joint than would the same loading applied to a similarly doweled expansion joint.

As mentioned earlier in this discussion of the effects of repetitive loading on load transfer, it is possible to relate data from the deflection measurements with those obtained from the strain measurements. In figure 34 the data from each type of measurement after 600,000 cycles with a 10,000-pound test load are compared. The increases in dowel looseness caused by the repetitive loading were determined from the relative deflection data, whereas the losses in initial load-transfer values were obtained from strain measurements. The computed relation was developed with formula 2 as described in the discussion of figure 23, the dowel looseness caused by the repetitive loading being substituted for initial looseness.

It may be concluded from figure 34 that there is an excellent correlation between the indications of the deflection data and those based on strain measurements on this important subject of the effects of repetitive loading on load transfer.

Some Dowel-Stress Data Obtained

Bradbury (*3*) and later Friberg (*4*) analyzed on the basis of theory the pressure, shear, and bending moment distributions developed in a steel dowel bar embedded in concrete, crossing an open joint, and acted upon by a load applied on one side of the joint. Recent experimental studies (*9*) have tended to verify the general validity of the earlier analyses.

When the present investigation was planned, no scheduled measurements of strains in the dowels or in the surrounding concrete were included. The measurement of strain either in the dowel or in the concrete, by the means presently available, usually involves the introduction of some disturbing modification in the materials or in the bearing of the dowels on the concrete that alters to some degree the basic condition being tested. It was for this reason that strain measurements at critical locations in the dowels or in the surrounding concrete were omitted from the scheduled measurements. However, a limited amount of strain data were obtained in specimens having a width of joint opening of ½ inch or greater by means of resistance type gages cemented to the dowel in the joint opening where no bearing contact was involved. It is recognized that these gages did not measure strains at the points of maximum bending moment. The data are of interest and some value, however, and are included in this report

The strain gages used were SR–4, type A–7, with an effective gage length of ¼ inch. They were bonded to the dowels along the upper element of the cylindrical dowel surface after the specimens were in place in the testing machine. The gages were used on each of the four dowels of a given specimen. In most cases the gage was so positioned that the center of its gage length was 5/32 inch from the vertical face of the concrete in which the dowel was embedded. A gage in this position provides two strain values during a given loading cycle: one value as the load is applied on that half of the specimen nearest the gage position, and a second value as the load is applied on the opposite half of the specimen.

Theory indicates that with a dowel installation of usual dimensions a vertical force applied outside of the encasing concrete should develop the maximum bending moment in the dowel at a point within the concrete a short distance back from the joint face. As mentioned earlier it would have been desirable to have had a strain gage at this point, but its presence would have affected other conditions more important in the current investigation. In the graphs and discussion which follow, measured strains have been converted to stress values using modulus of elasticity values determined by tests.

Figure 35 shows load-stress data obtained with a specimen having ¾-inch diameter dowels and a ¾-inch width of joint opening, when tested with a free-edge deflection rate of 0.01 inch per 1,000 pounds of applied load and before there had been any repetitive loading. The diagram shows also the details of the loading and strain gage positions with respect to the dowels.

The data in figure 35 are typical of those obtained in all of the dowel systems tested in a number of respects, as follows:

1. The load-dowel stress relations are essentially linear within the load range of 2,000 to 10,000 pounds. Departures from linearity at the lesser loads are believed to be due to initial adjustments in the seating of the dowels in their sockets.

2. The stresses in the two dowels nearest the loaded area are greater than those in the dowels farther from the load, indicating greater

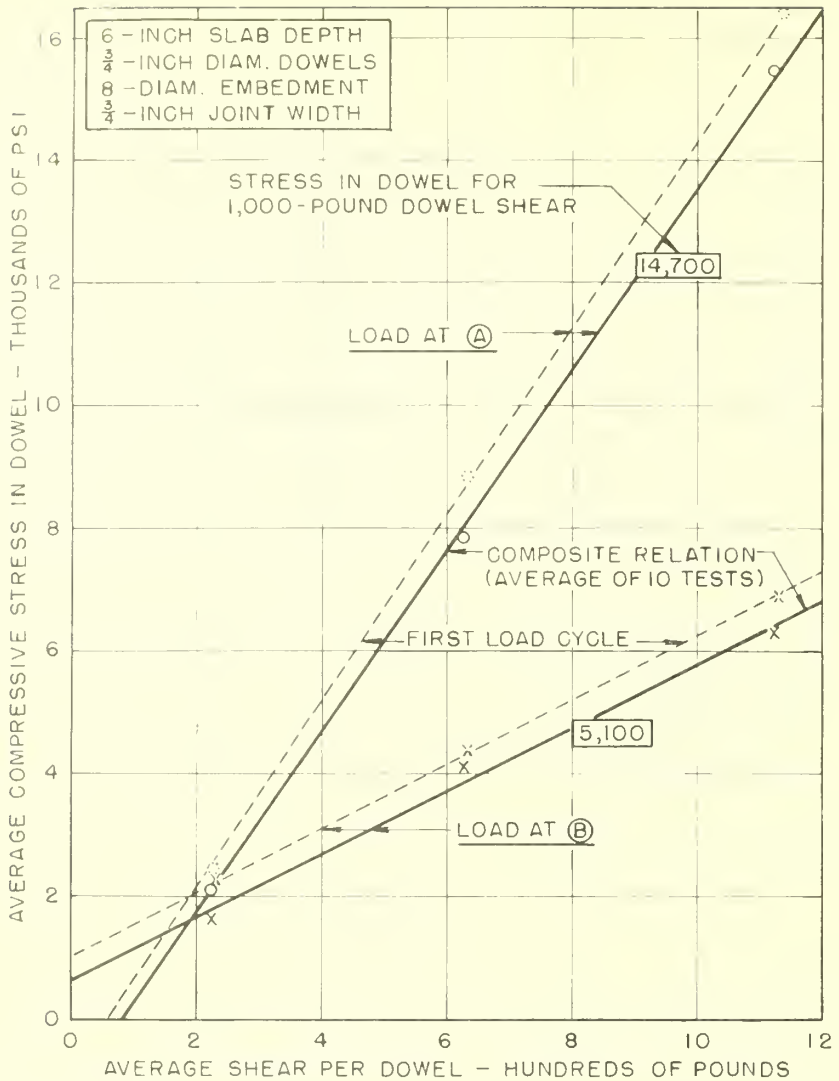

Figure 36.—Example of relations between average shear per dowel and average stress per dowel at gage point indicated in figure 35.

load transfer by these units as would be expected.

3. Within the joint opening the maximum stress in the dowel is found at the face of the loaded joint edge.

4. The point on the dowel within the joint opening at which the bending moment changes from positive to negative—the point of inflection of the elastic curve—was not found at the center of the joint width under the conditions of these tests. It will be shown later that the location of this point of inflection was found to vary with the physical dimensions of the dowels, the width of the joint opening, and with other conditions.

Although the stress values as determined from measured strains were not the maximum or critical values, they have been used in comparative studies of possible trends that might apply to the design variables of dowel diameter, length of dowel embedment, and width of joint opening.

A procedure was adopted which is believed to provide the best comparative data for the purpose. Essentially it consists of relating average values of dowel stress to average values of dowel shear in the manner shown in figure 36. In the example, the relation between the average shearing force per dowel and the average compressive stress in the dowel is shown for loads applied on either side of the joint opening. The relations are linear, so for each the slope is constant. The stress value per 1,000 pounds of shear force is shown as 14,700 p. s. i. with the load at point A, and 5,100 p. s. i. for the load at point B.

In figure 37 are diagrams representing joint widths of ½, ¾, and 1 inch, the diagram on the right being really a composite of three cases, one for each joint width. Average stress values, determined in figure 36, have been plotted in figure 37 as ordinates at distances from the joint faces corresponding to the respective points of strain measurement. The two points representing a given test specimen have been connected with a straight line which has been extended to meet the vertical lines representing the respective joint faces. These two points of intersection then indicate an estimated dowel-stress value at each joint face for a shear of 1,000 pounds. The values for the specimen used in figure 36 are shown as solid circles, whereas the open circles are values obtained in the same manner for other specimens. The values of estimated stress in the dowels at the face of the joint on the side bearing the load are those used in the following comparisons of the various dowel systems.

The diagrams of figure 37 show that within the joint opening the greatest stress occurs in the dowel at its point of entry into the concrete on the loaded side of the joint. This would be expected. Theory indicates that the stress should continue to increase in magnitude, rising to a maximum at some point a short distance from the joint face within the concrete.

Within the joint opening, the diagrams show that the stress—compression in the top and tension in the bottom of the dowel—decreases from its maximum value at the face of the loaded slab, the rate of decrease varying with the diameter of the dowel and the width of joint opening. In only two cases does the sense of the stress change (from compression to tension in the top of the dowel) within the joint opening. These cases are the ⅝-inch diameter dowels in the ¾-inch joint width and the ¾-inch diameter dowels in the 1-inch joint width. In each of these cases, a tensile stress of small magnitude is indicated at the face of the unloaded slab.

It is evident that the point of inflection in the elastic curve of the dowel depends upon the flexibility of the joint system; and, for the conditions of these tests, only in the case of the two most flexible systems was the point of inflection within the joint opening. Some additional information on this point is given later in the discussion of collateral tests.

Comparisons of Dowel Stresses

In figures 38–40 dowel-stress values at the face of the slab end on the loaded side of the joint, determined in the manner just described, are used to show the influence on dowel stress of the three design variables: dowel diameter, length of dowel embedment, and width of joint opening.

With a ¾-inch joint opening, an 8-diameter length of dowel embedment, and a dowel shear of 1,000 pounds, dowel stresses are compared in figure 38 for dowel diameters ranging from ⅝ inch to 1¼ inches. It is apparent that the relation expresses some inverse function of the dowel diameter. An empirical determination of the exponent required to produce a matching curve indicated its value to be 1.12. For the conditions of this test, it appears that increasing the diameter of the dowel from ⅝ inch to 1⅛ inches decreases the dowel stress at the joint face by about 50 percent. Had it been possible to compare values at the points of maximum stress, a somewhat greater reduction might have been shown.

The relations shown in figure 39 indicate that dowels having an embedded length of 2 diameters are stressed appreciably less than those with greater embedded lengths. The 2-diameter length of embedment is apparently insufficient to develop the full bending resistance of the dowels. It will be observed that beyond a certain length of embedment there is no further increase in dowel stress, and from the data available these lengths seem to

be in accord with the maximum useful lengths described earlier in the report.

As would be expected, the width of joint opening exerts an important influence on dowel stress, other conditions remaining constant. In figure 40 this effect is shown for a ¾-inch diameter dowel with an 8-diameter length of embedment. For these conditions a reduction in the width of joint opening from 1 inch to ½ inch reduced the dowel stresses more than 30 percent. These data emphasize the difference in severity of loading conditions imposed on dowels installed in expansion joints, as contrasted with similar units in contraction joints, and explain the generally better service performance of the latter.

It would be useful to know the magnitude of the maximum stresses which existed in the dowels during the repetitive loading program. For the reasons stated earlier these maximum values were not obtained. The estimated values at the joint face on the loaded side, while less than the true maxima, are of interest however. The highest individual values observed during the repetitive loading program for the several dowel diameters were as follows:

Dowel diameter, inches	*Dowel stress, p. s. i.*
⅝	27,200
¾	24,100
1	20,400
1⅛	15,500
1¼	13,700

Each value shown is the average of 10 measurements made during the application of 600,000 cycles of the 10,000-pound load.

During the repetitive loading program, the four ¾-inch diameter dowels in the pilot specimen were subjected to 2 million cycles during which the average maximum stress within the joint opening on the most highly stressed dowel was 21,600 p. s. i.

In another case, a specimen containing four 1¼-inch diameter dowels was subjected to 600,000 repetitions of the 10,000-pound load cycle during which the average maximum stress on the most highly stressed dowel was 13,700 p. s. i. This was followed by 500,000 cycles with a 15,000-pound load which developed a dowel stress of 18,600 p. s. i.

There was no apparent damage to the dowels as a result of the preceding tests. In only one special test did a dowel failure occur. This case is described later.

Figure 41 shows the oscillograph traces from strain gages mounted 5/32 inch from the joint face on each of the 4 dowels in tests of 2 specimens. In the record pertaining to the ¾-inch diameter dowels and the 1-inch width of joint opening, it is evident that in one case there was an actual stress reversal at the gage position as the load was changed from one side of the joint to the other. This is indicated in the upper trace by the fact that it crosses the horizontal base line from a high compressive strain (movement above the base line) to a small tensile strain (movement below the base line). Two of the other gages showed no strain, while the fourth gage showed a slight compressive strain during this half of the cycle.

In the case of the 1¼-inch diameter dowels and the ¾-inch width of joint opening, the indicated compressive strains are of lesser magnitudes than those of the ¾-inch dowels; unlike the latter, they are of very nearly equal magnitudes during both halves of the loading cycle.

The small vibrations indicated by that part of the trace which represents the full application of the load are caused by a slight elastic vibration set up in the loading lever-deadweight system as the lift-rod mechanism releases the load. Had the test been made at a higher frequency of load application, this effect would have become more pronounced unless some provision for damping were added to the system.

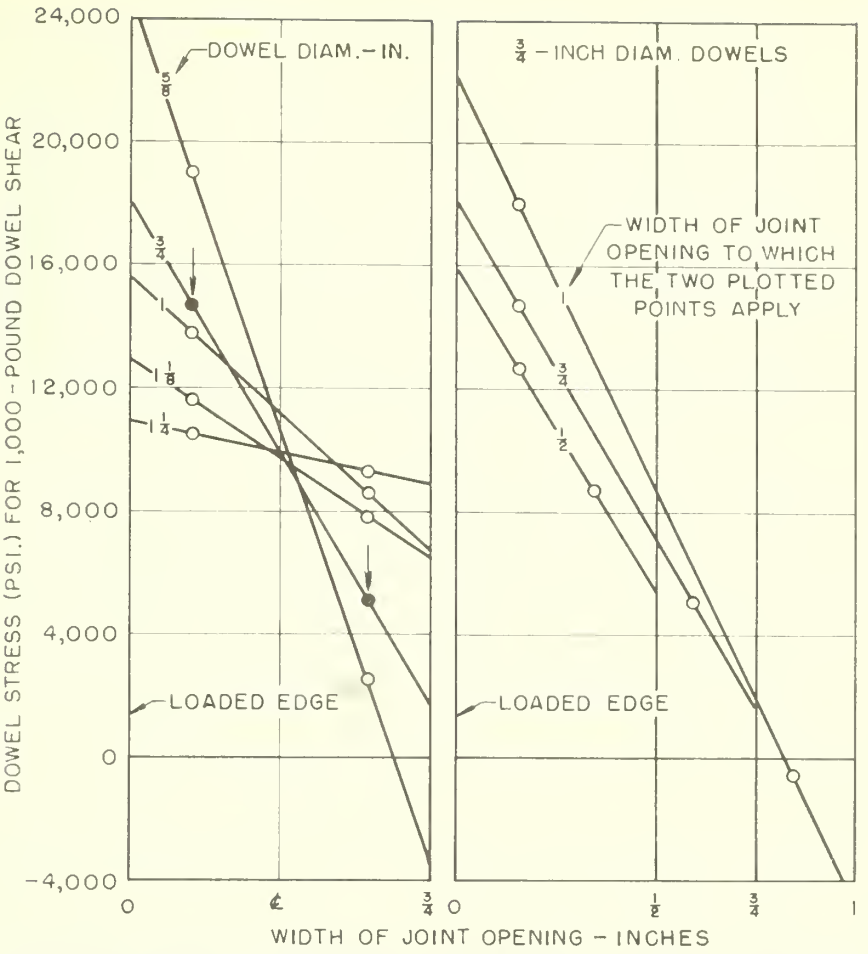

Figure 37.—Examples of distribution of dowel stresses within the width of joint opening.

Collateral Studies

In conjunction with the research program that has been described, a considerable amount of data of interest and value was obtained during the course of a number of collateral studies that were made. These data will be

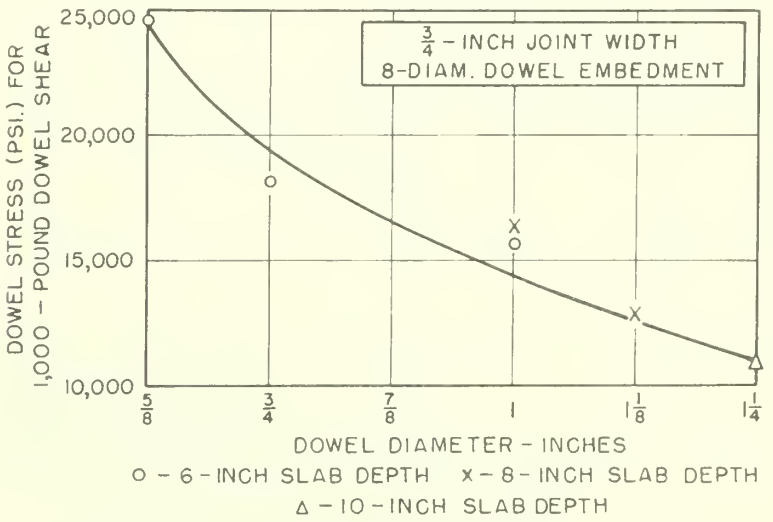

Figure 38.—Relation between dowel diameter and stress in dowel at face of loaded joint edge.

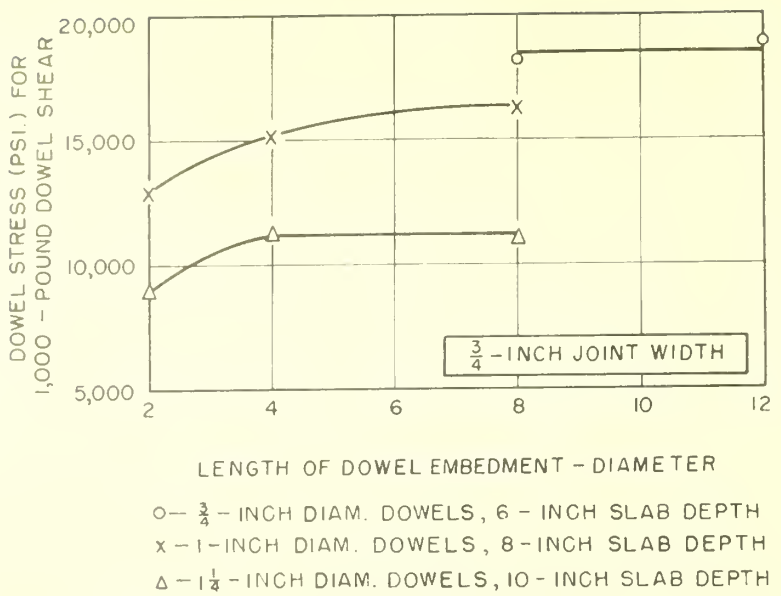
DOWEL STRESS (PSI.) FOR 1,000 - POUND DOWEL SHEAR
20,000
15,000
10,000
5,000
2
4
6
8
10
12
¾ - INCH JOINT WIDTH
LENGTH OF DOWEL EMBEDMENT - DIAMETER
O— ¾ - INCH DIAM. DOWELS, 6 - INCH SLAB DEPTH
X — 1 - INCH DIAM. DOWELS, 8 - INCH SLAB DEPTH
Δ — 1¼ - INCH DIAM. DOWELS, 10 - INCH SLAB DEPTH

Figure 39.—Relations between dowel embedment and stress in dowel at face of loaded joint edge.

discussed under appropriate headings in the paragraphs which follow.

Dowel performance related to pavement deflection

At several places in the report, mention has been made that 6 of the dowel systems were tested at a free joint-edge deflection rate of 0.02 inch per 1,000 pounds of applied load (or twice the deflection rate adopted for the primary program). This study was made on systems of ¾-inch diameter dowels embedded in slabs of 6-inch depth with a ¾-inch width of joint opening and various lengths of embedment. From the comparisons afforded by these data, it was found that the following conditions prevailed:

1. The percentage of load transferred was higher at the greater of the two deflection rates.

2. Values of the dowel deflection index (the deflection per 1,000 pounds of dowel shear) were in most cases slightly but only slightly higher for the greater of the two deflection rates.

3. Tests at both deflection rates indicated the same maximum useful lengths of dowel embedment.

4. For the same repetitive loading conditions, tests at the greater rate of joint-edge deflection developed greater dowel looseness.

5. As shown in figure 42, values of dowel stress per 1,000 pounds of dowel shear at the face of the loaded slab end averaged 21.5 percent higher for specimens tested at the greater of the two deflection rates. It is also indicated by this graph that as the deflection rate is decreased, the point of inflection on the dowel-stress distribution curve tends to move toward the center of the joint opening.

Comparison of 2 and 4 active dowels

In about one-half of the specimens tested, the outer dowels, or the dowels nearest the two sides of the specimens, were sawed through after completion of the scheduled test to provide some comparative data on the performance of 2- and 4-dowel systems.

It was found that among the systems tested the amount of load transferred by the 2 central dowels ranged from 97 to 99 percent of that transferred by 4 dowels. Thus it seems reasonable to infer that, other conditions being equal, the 4-dowel systems used in this investigation will transfer about the same proportion of the applied load as would the multi-dowel systems employed in the transverse joints of pavements in service.

The data afford two other interesting comparisons: First, with only 2 dowels active, the dowel-deflection index values tended in most cases to be slightly smaller than with 4 dowels active; and, second, as shown in figure 43, values of dowel stress per 1,000 pounds of dowel shear were appreciably reduced by severing the 2 outer dowels.

Flexural stresses in concrete

In the early planning of the tests it was considered desirable to develop some bending in the concrete along the joint edge to simulate more closely the conditions that would be found in the field. The early tests were made with a 6-inch depth of specimen, and by trial the length of bearing between the slab and the support beam was adjusted to develop a bending stress of about 300 p. s. i. with the 10,000-pound test load. Later when slabs of 8- and 10-inch depth were tested, it was decided to maintain the same length of bearing throughout.

For the constant test load and rate of free-edge deflection, the effect was to reduce the bending stress in the concrete as the slab depth was increased approximately as the reciprocal of the depth squared. Figure 44 shows the position of the strain gage with respect to the area of load application as well as typical data obtained with specimens of the three thicknesses. The gage, cemented to the concrete, was the SR–4, type A–9, with an effective length of 6 inches. It is apparent for the conditions of the test that the behavior of the concrete was elastic.

Concrete bearing stresses

Compressive stresses in the concrete in the vicinity of the dowels exert an important influence on the structural performance of load-transfer systems. In the case of the conventional round steel dowel, it has long been recognized that the compressive stresses above and below the dowel may be critically high. Marcus (*10*) has published experimental data which indicate bearing stresses under dowels, the magnitude of which was more than twice the compressive strength of the concrete.

These stresses are a maximum at the face of the slab end and are concentrated immediately above or below the dowel, depending upon the side of the joint on which the load is acting.

This investigation included no provision for a study of this important subject. However, on one specimen of 8-inch depth, an installation of three resistance strain gages with an effective length of ¼ inch was made in the

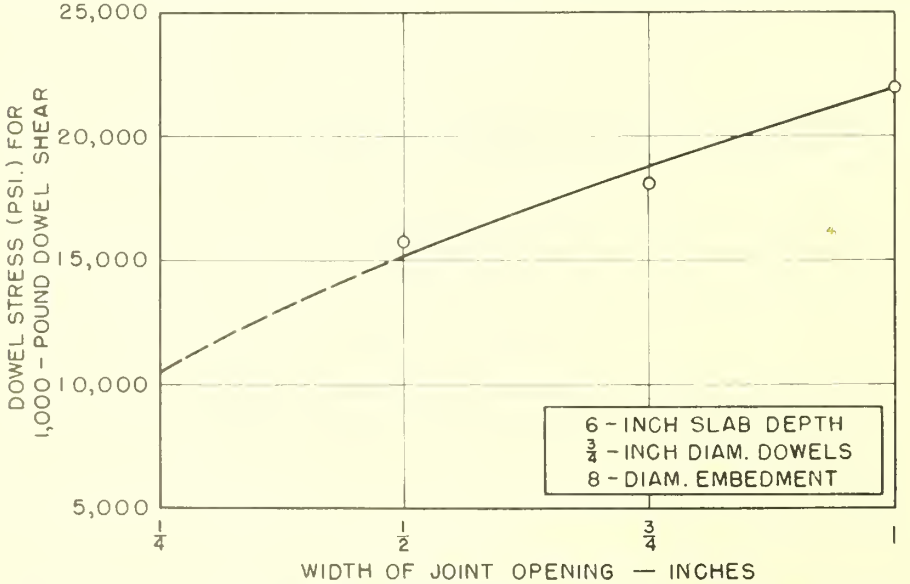

Figure 40.—Relation between width of joint opening and stress in dowel at face of loaded joint edge.

manner shown in figure 45. The load-strain relations obtained are those shown in the diagram. These are linear within the range of the test.

Values of concrete strain measured over so short a gage length may be affected to some extent by non-homogeneity of the concrete in the immediate vicinity of the gages. For this reason the data are expressed as measured strains. They are of considerable interest, however, as they indicate clearly the high intensity and localized effect of the pressure exerted by the dowel.

From the slopes of the load-strain relations of the three strain gages, values of concrete strain for 1,000 pounds of dowel shear were obtained and shown in relation to the distance from the top of the dowel in figure 46. This diagram indicates the highly localized nature of the deformation that occurs in the concrete and is in general accord with previously reported data.

Distribution of dowel stress

In the tests of the regular program, only one strain gage was attached to a dowel because of the limited space within the joint openings. As explained earlier this led to an interpretation of a linear distribution of dowel stress between the joint faces. To obtain some data that would show whether or not this was a valid interpretation, the joint of one of the specimens was opened to a 3½-inch width after the completion of the prescribed tests. This gave sufficient space to permit the installation of five strain gages along the dowel. The specimen selected was 10 inches in depth and the dowels were 1¼ inches in diameter.

The location of the gages and the data obtained in this test are shown in figure 47. The two curves show load-stress relations within the joint opening for loadings at points A and B on either side of the joint opening. From this diagram it may be concluded that within the limits of the test program the dowel-stress distribution across the joint opening is linear. It is of interest also that by opening the joint to 3½ inches, the flexibility of the system was increased to such an extent that the point of inflection or zero stress of these 1¼-inch diameter dowels appeared within the joint opening.

Modulus of dowel reaction

Extrapolating the dowel deflection data obtained with the various widths of joint opening (figure 14) beyond the 1/16-inch width to an assumed zero width, and using the mechanical properties of the steel and concrete as determined by test, values of the so-called modulus of dowel reaction were computed for the 6-inch slab depth and ¾-inch diameter dowel, the 8-inch slab depth and 1-inch diameter dowel, and the 10-inch slab depth and 1¼-inch diameter dowel. The values obtained were 3,026,000, 2,608,000, and 2,675,000 pounds per cubic inch, respectively.

The modulus of dowel reaction or modulus of support is a measure of the support offered a dowel by the surrounding concrete as the dowel is deflected. It is analogous to the subgrade modulus used to express the support

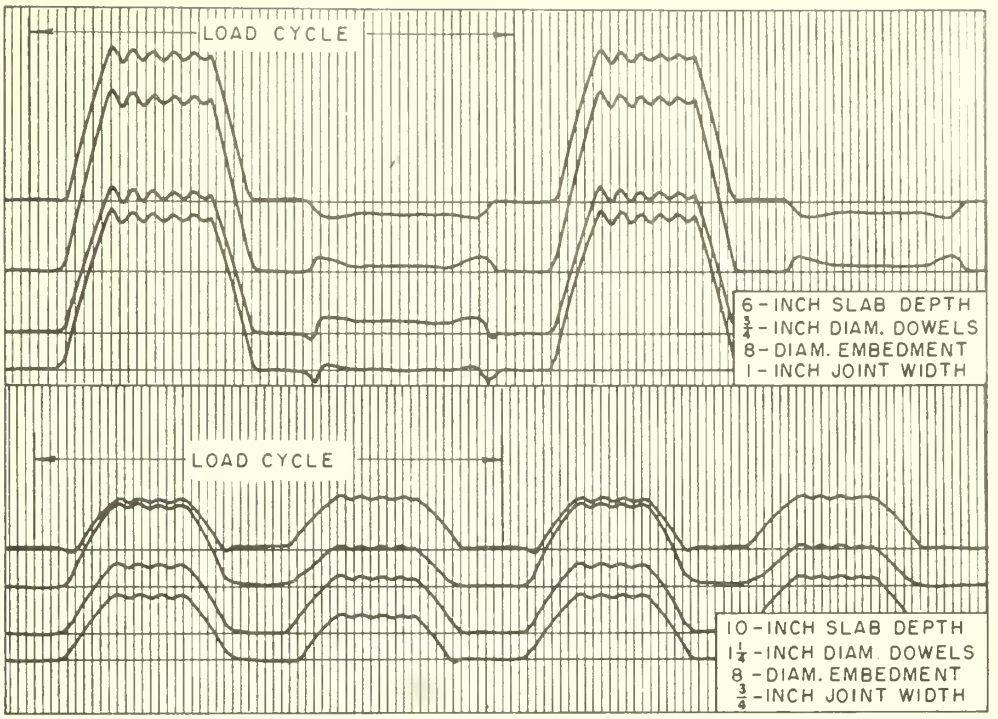

Figure 41.—Oscillograms of strain in individual dowels for a 10,000-pound load.

afforded the pavement slab by the subgrade, and like the latter it is expressed in pounds-per-cubic-inch units. It is usually denoted by K or G in the literature.

Fatigue failure of steel dowels

It was stated earlier that in none of the tests of the regular program was there a failure of any of the steel dowels, in spite of the relatively high flexural stresses and the relatively large number of stress reversals in some of the tests. In one special test a fatigue failure of the dowels was produced, and both the procedure followed and the character of the failure are of some interest.

A specimen containing four ¾-inch diameter dowels which had undergone 600,000 cycles with a 10,000-pound load was selected for further testing. During the regular testing, with 4 dowels active, the bending stresses at the joint face of the 2 central dowels averaged 18,800 and 22,800 p. s. i., respectively.

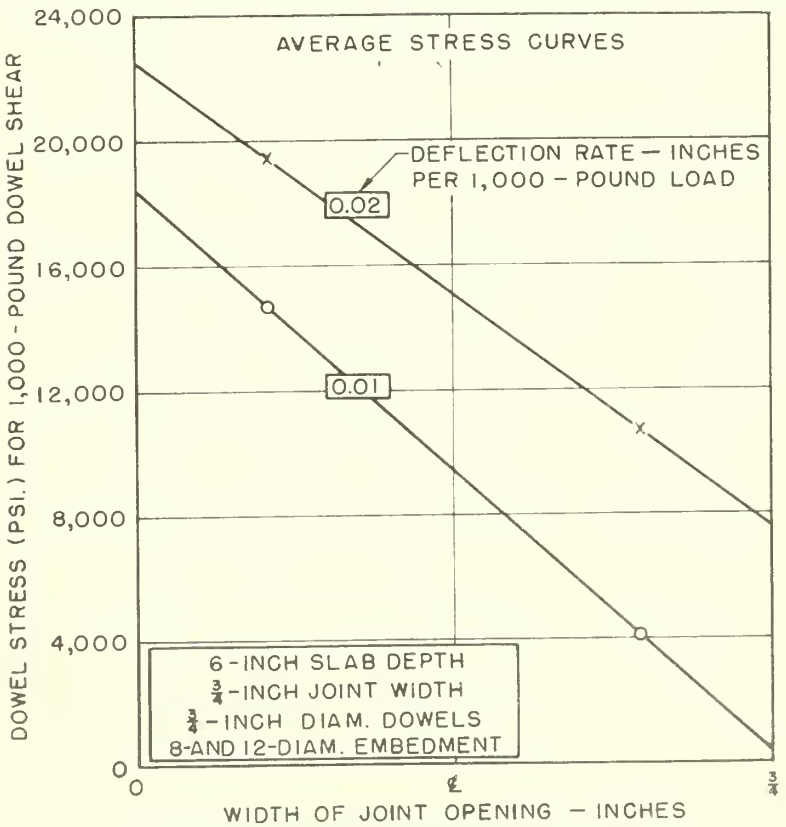

Figure 42.—Effect of deflection rate on dowel stress.

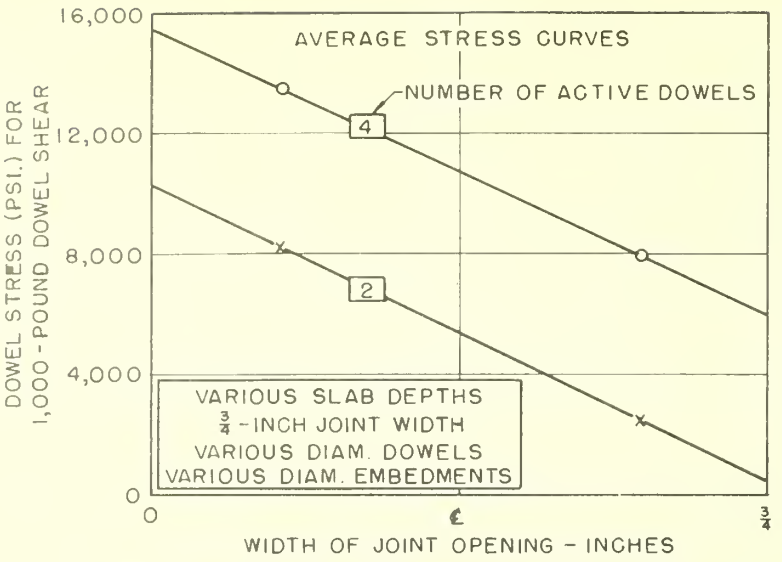

Figure 43.—Effect of number of active dowels on dowel stress.

The 2 outer dowels were severed and repetitive loading was resumed still using the 10,000-pound load. With but 2 dowels active the indicated stresses in the dowels increased to 24,300 and 28,200 p. s. i., respectively. After 892,000 additional cycles of loading, failure occurred in both dowels. This happened outside of regular working hours and the sequence of events can only be surmised. The breaks were brittle fractures typical of fatigue failures. The dowel that was being stressed most highly failed on both sides of the joint opening; the other dowel failed on one side only.

Figure 48 is a photograph of the central piece of the dowel that broke on both sides of the joint. The two end views show the character of the fractures; the side view shows the location of the breaks with respect to the joint opening. The location of the breaks undoubtedly indicates the point of maximum moment in the dowel. In this case the fractures appear to be about ½-dowel diameter back from the joint face, within the concrete, on one side of the joint and approximately 1-dowel diameter on the other side of the joint. The location of break in the dowel which failed on one side of the joint opening only was within the concrete about ½-dowel diameter from the joint face.

From these failures it is apparent that the estimated dowel-stress values at the face of the joint (or slab end), arrived at by measurements of dowel strains within the joint opening, are probably appreciably less than the corresponding values at the point of maximum bending moment somewhere along the embedded length. In the case of the ¾-inch diameter dowels, this point would appear to be not less than ½-dowel diameter from the beginning of the embedment.

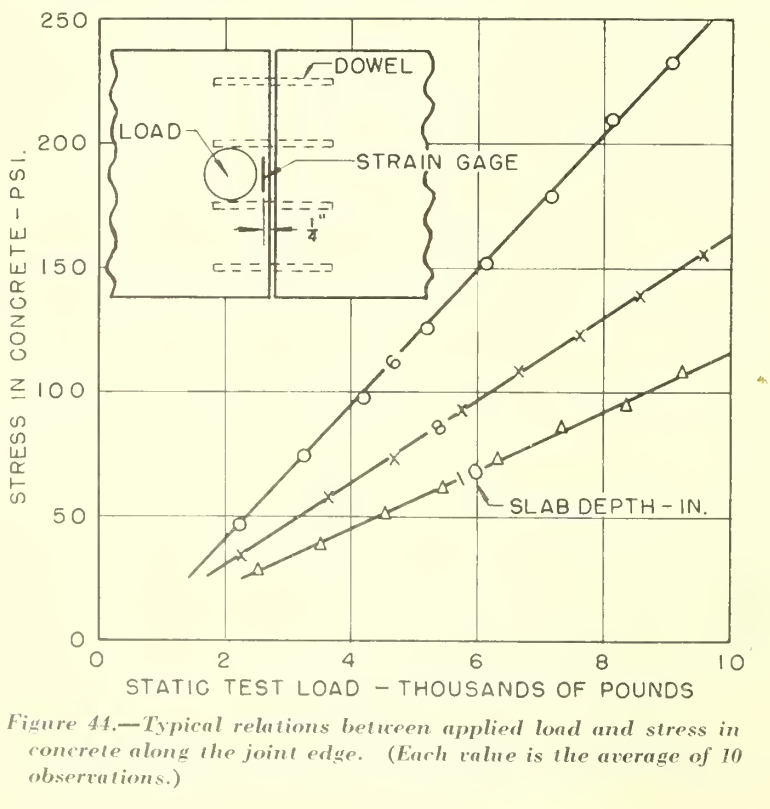

Figure 44.—Typical relations between applied load and stress in concrete along the joint edge. (Each value is the average of 10 observations.)

Area of Further Research

The present test program has developed much important information on the performance of conventional round steel dowels under repetitive loading conditions which simulate closely those of actual service. This initial program could not include all of the variable factors involved in such service, however, and there are important questions that can be answered only by further tests. For example, the tests to date have all been made with high strength concrete in a dry condition. A limited program of additional tests is needed to relate the present data to those obtained with concrete of somewhat lower strength and containing entrained air. Also some tests should be made with concrete in a moist condition such as that found in most field service.

It would appear that this test procedure might be used effectively to develop much needed information on the perplexing question of load transfer by the fractured surfaces of plane of weakness joints in which there is no provision for mechanical load transfer.

In the program of tests just completed, tests were made with joint deflections which were representative of weak subgrade support. A limited series of additional tests should be made to relate the present data to other conditions of subgrade support, particularly those of a firm subgrade support.

Because of the high intensity of bearing pressure that is known to develop between the conventional dowel and the concrete, various expedients have been proposed for reducing this pressure. For example, the use of tubular dowels is a suggestion that has been made from time to time. The repetitive loading test procedure offers a means for evaluating this and other proposals of a generally similar nature.

Some load-transfer devices are made of malleable cast iron, a material that is susceptible to permanent deformation under repeated stressing. Some study of the behavior of malleable cast iron, when subjected to repetitive loading under conditions comparable to highway pavement service, should provide valuable information.

The method of test that has been developed and the machines for making the test provide a valuable facility for studying a number of other problems relating to the structural performance of joints.

Appendix

Friberg equations

Modulus of dowel reaction.—From measurements of the deformation of the concrete, y_o, at the face of the joint, under the bearing load, P, exerted by the dowel, a constant term, β, is obtained by utilizing the expression:

$$y_o = \frac{P - \beta M_o}{2\beta^3 E_s I}$$

In which:

M_e = The moment in the dowel at the face of the joint caused by the load, P, acting at a distance equal to half the width of the joint opening or $\frac{a}{2}$ (inch-pounds).

E_s = The modulus of elasticity of the dowel steel (p. s. i.).

I = Moment of inertia of the dowel section (inches^4).

Having the value of β, the modulus of dowel reaction, K, may be obtained from the expression given by Friberg for the relative stiffness of the structure (dowel) and the mass (concrete), as follows:

$$\beta = \sqrt[4]{\frac{Kb}{4E_sI}}$$

The letter b represents the diameter of the dowel (in inches) and the other terms are as previously defined. The modulus of dowel reaction is expressed in pounds-per-cubic-inch units.

Dowel deflection.—The deflection of a dowel crossing a joint may be determined from the following equation:

$$\Delta = \frac{P}{2E_sI}\left(\frac{1+(1+\beta a)^2}{\beta^3} + \frac{a^3}{6}\right)$$

In which:

Δ = The difference in deflection of the loaded and unloaded sides of the joint with the dowel in full bearing on the concrete (in inches). The other terms are as previously defined.

Westergaard equations

Slab deflection.—The deflection of a free slab edge and corner may be determined from the following equations:

$$z_e = \frac{0.433\,P}{kl^2}$$

$$z_c = \left(1.1 - 0.88\frac{a_1}{l}\right)\frac{P}{kl^2}$$

In which:

z_e and z_c = Maximum deflection for edge and corner loadings, respectively (in inches).

P = Applied load (pounds).

k = Modulus of subgrade reaction (p. c. i.).

a_1 = A dimension, measured along the bisector of the corner angle in the case of corner loading and equal to the diameter of the loaded area, a, multiplied by $\sqrt{2}$.

The dimension, l, termed the radius of relative stiffness, measured in inches, may be determined from the following expression:

$$l = \sqrt[4]{\frac{E_ch^3}{12\,(1-\mu^2)k}}$$

In which:

E_c = Modulus of elasticity of the concrete (p. s. i.).

h = Depth of the slab (inches).

μ = Poisson's ratio for concrete.

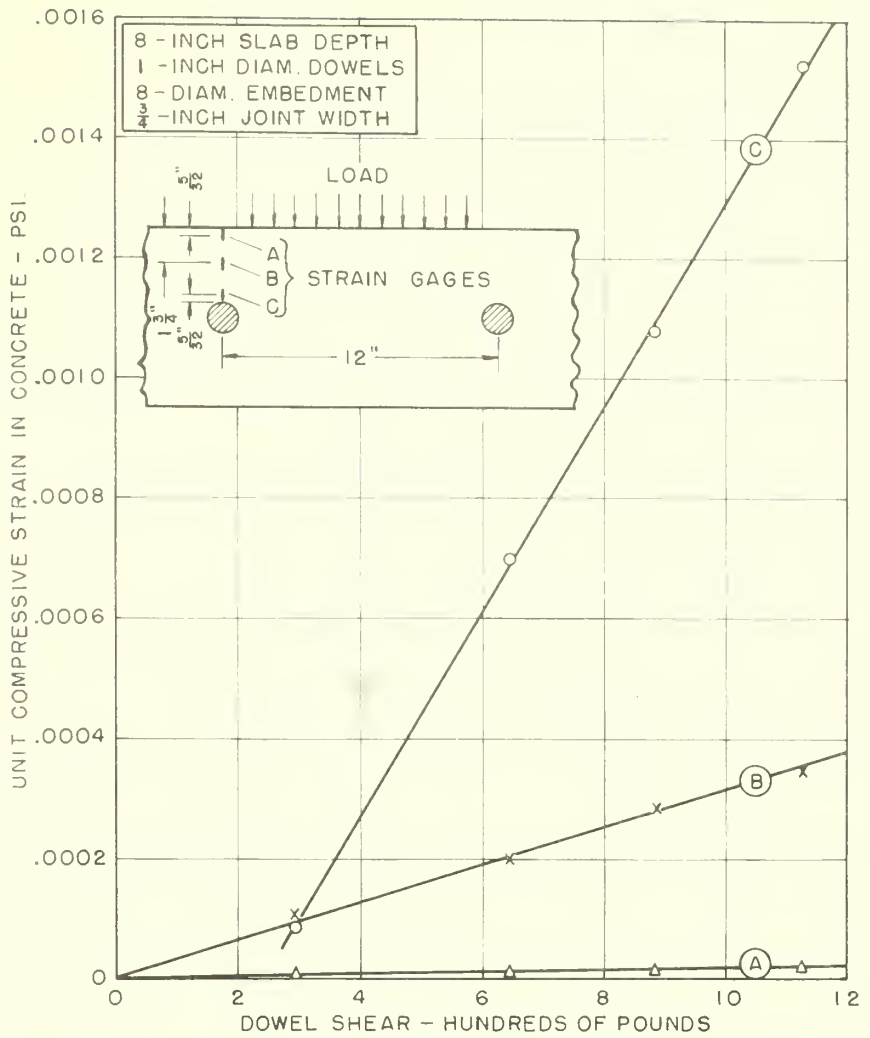

Figure 45.—Dowel shear-concrete strain relations for the indicated points (A, B, and C) at the face of the loaded joint edge. (Each value is the average of 10 observations.)

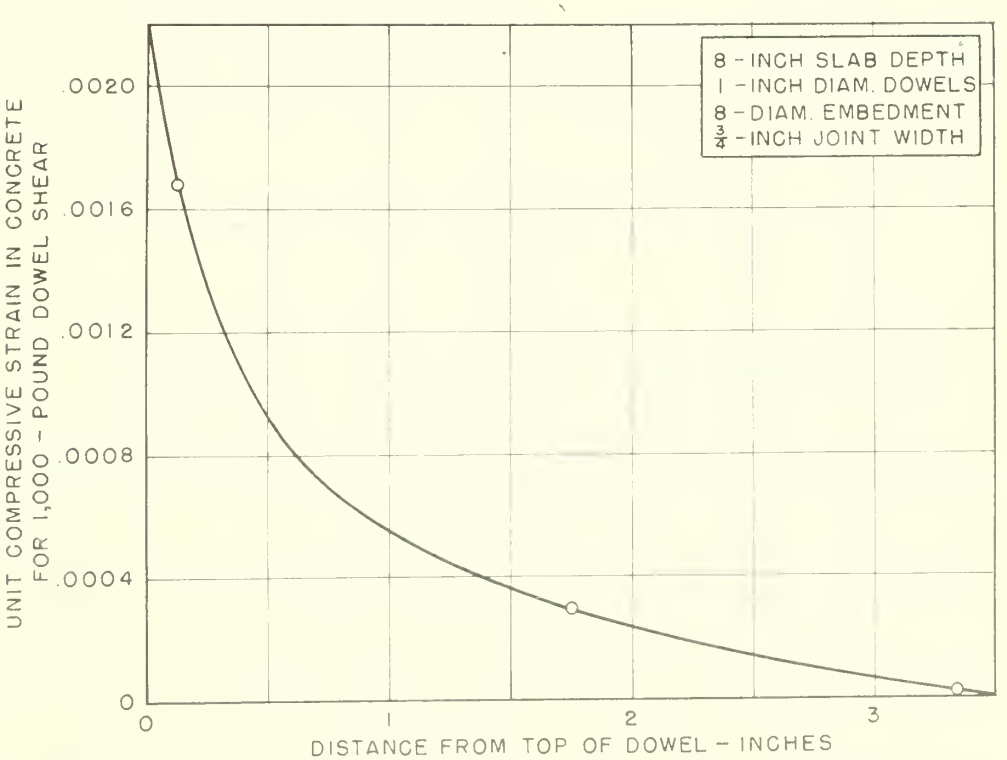

Figure 46.—Variation in bearing strains in the concrete directly above the dowel, as measured at the face of the loaded joint edge.

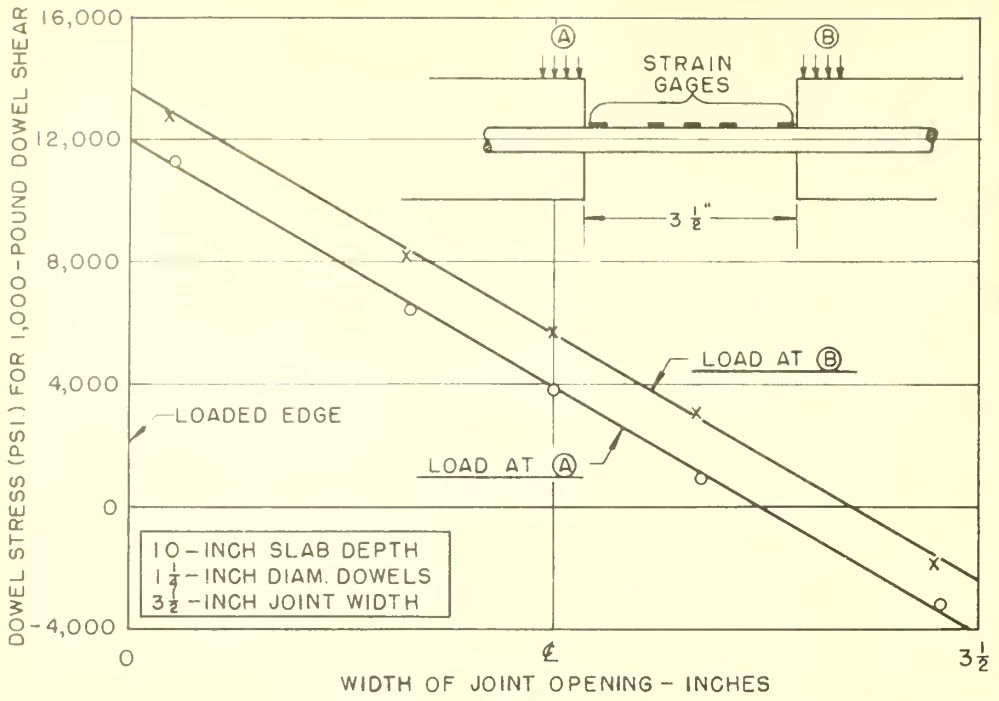

Figure 47.—Distribution of dowel stresses within the width of a 3½-inch joint opening.

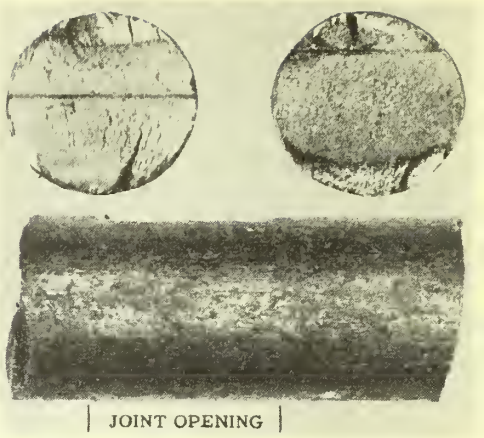

Figure 48.—Fatigue failure of a ¾-inch diameter dowel. The cross-section views at the top correspond to the left and right ends of the fractured dowel.

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New Publications

The Secretary of Commerce transmitted to the Congress on January 7, 1958, two reports of studies made by the Bureau of Public Roads in cooperation with the several State highway departments.

A Report of Factors in Apportioning Funds for the National System of Interstate and Defense Highways, prepared pursuant to section 108 (d) of the Federal-Aid Highway Act of 1956, has been published as House Document No. 300, and is available from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at 15 cents a copy.

The second report, *Consideration for Reimbursement for Certain Highways on the Interstate System*, prepared in accordance with section 114 of the 1956 Act, has been published as House Document No. 301, and is available from the Superintendent of Documents at 15 cents a copy.

Also available is the *Annual Report of the Bureau of Public Roads, Fiscal Year 1957*, which may be purchased from the Superintendent of Documents at 30 cents a copy.

The 85-page publication—57 pages of text and 28 pages of statistical tables—describes the operation of the Federal-aid highway program for the fiscal year ended June 30, 1957. Included is a comprehensive review of the Bureau's accomplishments in highway construction, foreign assistance programs, and research in roadbuilding materials, highway transport, hydraulics, and finance and administration.

Errata

The table *State Legal Maximum Limits of Motor Vehicle Sizes and Weights Compared With AASHO Standards*, which appears on pages 256–257 of Public Roads magazine, vol. 29, No. 11, December 1957, requires a correction in the numbering of footnotes.

The practical maximum gross weight of 3-axle trucks (sixth column from the right) in Michigan (40,000 pounds) should carry footnote 35 instead of 30.

U. S. GOVERNMENT PRINTING OFFICE: 1958

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A Report of Factors for Use in Apportioning Funds for the National System of Interstate and Defense Highways, House Document No. 300 (1958). 15 cents.

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1946 (out of print). 1950 (out of print). 1954, 75 cents.
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1948, 65 cents. 1952, 75 cents.

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Highways in the United States, *nontechnical* (1954). 20 cents.

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Progress and Feasibility of Toll Roads and Their Relation to the Federal-Aid Program, House Document No. 139 (1955). 15 cents.

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Public Land Acquisition for Highway Purposes (1943). 10 cents.

Public Utility Relocation Incident to Highway Improvement, House Document No. 127 (1955). 25 cents.

Results of Physical Tests of Road-Building Aggregate (1953). $1.00.

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Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1956: a reference guide outline. 55 cents.

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STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF FEBRUARY 28, 1958

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$33,593	$57,838	$43,645	451.7	$18,088	$11,758	158.0	$78,368	$52,057	770.1	$154,294	$107,460	1,379.8
Arizona	10,458	27,675	24,463	146.1	10,551	9,586	60.3	33,940	29,104	202.4	72,166	63,153	408.8
Arkansas	24,408	42,837	33,071	554.6	18,566	13,556	124.3	49,804	35,683	355.5	111,207	82,310	1,034.4
California	55,623	36,339	21,544	100.4	56,400	47,693	53.7	565,067	246,905	393.6	657,806	316,142	607.7
Colorado	37,556	16,862	12,708	102.9	3,326	1,829	25.7	57,259	41,900	300.7	77,447	56,437	429.3
Connecticut	51,502	2,358	1,296	.9	3,798	2,497	1.5	42,246	29,126	36.0	48,402	32,919	38.4
Delaware	22,766	12,183	9,124	22.7	2,439	1,228	3.7	12,917	7,528	61.7	27,539	17,880	88.1
Florida	10,948	70,624	54,146	307.6	6,181	4,327	33.8	69,205	48,143	350.0	146,010	106,616	691.4
Georgia	18,524	120,159	91,786	694.9	21,196	14,842	101.0	102,133	58,720	934.6	243,488	165,348	1,730.5
Idaho	37,269	18,263	15,106	99.1	3,058	1,844	46.6	20,501	14,679	161.9	41,822	31,629	307.6
Illinois	46,465	100,468	69,425	553.0	79,080	60,262	143.9	201,479	151,058	620.9	381,027	280,745	1,317.8
Indiana	94,949	65,072	45,391	270.3	26,853	14,966	269.8	59,471	38,200	264.5	151,396	98,557	804.6
Iowa	30,987	54,969	41,054	686.3	28,662	22,722	179.3	58,068	39,221	1,190.8	141,699	102,997	2,056.4
Kansas	27,204	28,593	21,470	723.8	30,541	22,745	324.0	57,025	39,232	1,190.4	116,159	83,447	2,238.2
Kentucky	49,777	43,275	29,235	139.6	3,434	2,065	17.0	71,495	49,794	242.1	118,204	81,094	398.7
Louisiana	19,339	68,451	49,273	295.8	34,088	23,729	132.2	66,185	39,362	379.3	168,724	112,364	807.3
Maine	26,494	11,243	9,130	17.5	4,836	3,866	4.4	21,802	13,895	104.4	37,881	26,891	126.3
Maryland	9,971	22,677	13,854	106.2	32,153	21,135	56.6	72,172	51,778	172.6	127,002	86,767	335.4
Massachusetts	28,655	48,357	39,285	25.0	43,205	25,144	22.9	105,082	69,938	57.1	196,644	134,367	105.0
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Mississippi	12,280	50,771	36,788	691.2	28,391	22,482	188.5	62,630	43,195	771.5	141,792	102,465	1,651.2
Missouri	31,828	50,784	35,801	1,155.9	22,201	16,573	78.6	106,538	70,740	1,012.8	179,523	123,114	2,247.3
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New Jersey	85,930	20,453	14,138	53.4	8,001	6,570	5.7	76,259	54,000	51.3	104,713	74,708	110.4
New Mexico	9,645	11,159	9,286	37.9	7,529	5,985	62.1	46,601	37,658	296.9	65,289	52,929	396.9
New York	126,712	50,705	36,291	103.5	66,014	51,698	32.6	507,069	321,239	394.3	623,788	409,228	530.4
North Carolina	65,240	51,755	39,953	335.0	11,766	8,085	82.1	84,921	55,606	741.9	148,442	103,644	1,159.0
North Dakota	18,461	27,645	20,697	905.3	18,248	14,528	250.1	24,901	16,991	985.1	70,794	52,216	2,140.5
Ohio	16,994	65,135	35,637	156.1	74,064	61,710	73.2	244,627	169,957	330.6	383,826	267,304	559.9
Oklahoma	16,193	71,066	51,532	694.7	15,051	11,102	67.0	72,353	48,418	585.4	158,470	111,052	1,347.1
Oregon	31,200	7,790	5,942	53.9	13,359	10,618	68.3	39,911	31,571	194.2	61,060	48,131	316.4
Pennsylvania	100,672	121,647	85,367	186.6	35,497	21,879	64.6	261,572	175,410	361.3	418,716	282,656	612.5
Rhode Island	9,440	13,715	10,346	16.7	3,096	2,666	1.6	24,413	17,297	21.0	41,224	30,309	39.3
South Carolina	21,594	63,459	49,099	543.2	3,258	2,128	36.2	47,176	29,723	673.0	113,893	80,950	1,252.4
South Dakota	3,921	70,010	56,382	881.3	1,538	860	57.0	21,792	15,783	445.3	93,340	73,025	1,383.6
Tennessee	34,202	75,628	57,617	555.2	24,907	17,056	71.1	90,102	55,686	537.2	190,637	130,359	1,163.5
Texas	62,767	128,492	108,807	651.1	47,720	34,652	156.5	224,632	160,514	1,893.9	400,844	303,973	2,701.5
Utah	7,022	33,195	30,159	190.5	8,291	7,067	36.4	22,711	19,144	107.6	64,197	56,370	334.5
Vermont	4,961	22,938	19,289	43.8	1,765	882	9.0	19,698	14,821	38.4	44,401	34,992	91.2
Virginia	18,632	53,112	41,687	137.7	20,965	15,728	43.4	83,863	60,733	261.2	157,940	118,148	442.3
Washington	36,454	26,641	18,932	230.9	6,778	4,185	15.7	63,241	47,772	217.5	96,660	70,889	464.1
West Virginia	39,676	45,435	33,688	71.6	3,927	1,975	14.7	38,948	21,905	71.5	88,310	57,568	157.8
Wisconsin	80,960	27,638	18,015	322.8	18,640	12,655	142.4	47,091	30,094	375.9	93,369	60,764	841.1
Wyoming	6,384	22,327	18,318	151.3	2,485	2,105	23.9	45,906	38,359	266.6	70,718	58,782	441.8
Hawaii	2,835	10,617	5,309	17.2	1,488	689	1.8	3,523	1,718	4.4	15,628	7,716	23.4
District of Columbia	24,175	6,793	4,745	4.8	5,227	3,390	2.8	25,652	19,423	4.5	37,672	27,558	12.1
Puerto Rico	7,589	18,147	8,789	28.8	1,024	478	5.6	18,769	8,678	55.3	37,940	17,945	89.7
Alaska	13,809	3,419	3,419	46.3	617	617	11.0	10,800	9,292	307.9	14,836	13,328	365.2
TOTAL	1,816,214	2,143,270	1,589,556	14,731.4	942,085	685,682	3,917.0	4,426,938	2,900,593	21,530.0	7,512,293	5,175,831	40,178.4

VOL. 30, NO. 2

JUNE 1958

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: A discussion of new methods for determining traffic capacity of rural highways.
(U. S. Route 40, south of Hagerstown, Md.)

Public Roads

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Published Bimonthly

Vol. 30, No. 2 *June 1958*

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IN THIS ISSUE

New Methods for Determining Capacity of Rural Roads in Mountainous Terrain 25

The Economic Costs of Motor-Vehicle Accidents of Different Types 10

Use of the Swiss Hammer for Estimating the Compressive Strength of Hardened Concrete . 15

Wind Tunnel for Aerodynamic Testing of Section Models of Suspension Bridges 51

Power Shovel Productivity—a motion picture . 52

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

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Under Secretary for Transportation

BUREAU OF PUBLIC ROADS
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New Methods for Determining Capacity of Rural Roads in Mountainous Terrain

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by O. K. NORMANN, Chief, Traffic Operations Branch, Bureau of Public Roads, JAMES O. GRANUM, Automotive Safety Foundation, and HARRY C. SCHWENDER, West Virginia State Road Commission

The development of traffic capacity information needed for the design of roads in rough or mountainous terrain has required the combined results of a number of independent yet related investigations, including studies of commercial-vehicle weights and performance characteristics, driver passing practices, and the many types of studies normally associated with a capacity analysis such as frequency distributions of speeds and headways under different traffic volume and roadway conditions.

This article is a combined effort of several agencies to develop practical procedures for the application of the results of such studies to a determination of highway needs in mountainous terrain. The procedures are, however, equally applicable to all types of terrain. They provide a significant advance in analytical procedures for determining improvements in traffic-operating conditions through better alinement, reduction in gradient, and the use of truck climbing lanes on the uphill side of steep grades.

IN a comprehensive survey of highway needs, it is essential to establish a certain level of performance for each highway system in a State and then determine the improvements needed to bring the existing systems up to these performance levels. The levels of performance which are established must be feasible and be based on safe and efficient operation of vehicles, with due consideration being given to the future demands of highway transportation.

Performance levels may be measured and also specified in terms of safe operating speeds. Comprehensive studies as well as past experience have shown that drivers demand, and that it is more feasible to construct, facilities that will permit higher operating speeds (1) in level terrain than in rough or mountainous terrain, (2) on primary highways carrying most of the long-distance travel than on local roads where the average trip length is shorter, and (3) on roads of the same system carrying the higher traffic volumes than on those carrying the lower traffic volumes. While the type of service to be provided by a highway under construction is largely an administrative decision, this decision must be based on driver desires and traffic demand.

Once having established the type of service which a highway or a system of highways should provide, it is necessary to specify this service in terms of design speed and operating speed. The *design speed* is a speed determined for design and correlation of the physical features of a highway that influence vehicle operation; it is the safe speed that can be maintained over a specified section of the highway when conditions are so favorable that the design features of the highway govern (*1*).[2] In short, it is the maximum speed that vehicles can safely travel over any section of the highway during extremely low traffic densities. In the design of a highway, the assumed design speed automatically establishes such features as the minimum stopping sight distance, the minimum sight distance at intersections, the maximum curvature, and the superelevation. Other features, such as widths of pavements and shoulders and clearances to walls and rails, are not directly related to design speed but they should be accorded higher standards for the assumed higher design speeds.

The *operating speed* is the highest overall speed exclusive of stops at which a driver can safely travel on a given highway under the prevailing traffic conditions without at any time exceeding the speed which is compatible with the design features of the highway. For this discussion it applies to the conditions during the 30th highest hourly traffic volume for the year under consideration. It is therefore a measure of the type of service which a highway provides during most of the hours of peak flow. The operating speed on an existing highway is affected by the design speed, the traffic volume, and the number of lanes. Also, for two-lane roads, it is affected by the availability of sections on which the sight distance is of sufficient length to permit safe passing maneuvers. In the design of a new highway, it is the one factor which together with the traffic volume and assumed design speed determines the needed geometric features.

Drivers will accept as reasonable a somewhat lower operating speed, or a higher degree of congestion, on a highway that has been in existence for several years than they will accept or expect on a new highway or one recently reconstructed. Also for a needs study to be realistic, there must necessarily be some overlap in the standards by which existing highways are judged for adequacy and those used for a new highway.

In the early stages of the West Virginia Highway Needs Study, the Engineering Committee after reviewing the results of speed studies on highways throughout the State agreed upon a set of tolerable conditions for judging the adequacy of existing highways in order to determine those in need of construction or reconstruction. A set of standards was also prepared for use on new construction. Both were in terms of operating speeds and design speeds. The tolerable conditions and the construction standards for highways in West Virginia carrying over 1,800 vehicles per day are shown in table 1. These conditions and standards were determined prior to the passage of the Federal-Aid Highway Act of 1956.

After these tolerable conditions and standards in terms of service to traffic had been

[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957. Mr. O. K. Normann was appointed Deputy Assistant Commissioner, Office of Research, effective September 8, 1957.

[2] Italic numbers in parentheses refer to the list of references on p. 44.

Table 1.—Tolerable conditions for existing rural highways in West Virginia carrying over 1,800 vehicles per day, and standards for new construction or reconstruction in terms of the service provided [1]

Highway system and type of terrain	Tolerable conditions		Construction standards	
	Operating speed	Design speed	Operating speed	Design speed
	M. p. h.	*M. p. h.*	*M. p. h.*	*M. p. h.*
Interstate System highways:				
Valley or level terrain	45–50	60	50–55	70
Rolling terrain	40–45	50	45–50	60
Mountainous terrain	40–45	45	45–50	60
Other than Interstate System highways:				
Valley or level terrain	45–50	60	50–55	70
Rolling terrain	40–45	50	45–50	60
Mountainous terrain	35–40	40	40–45	60

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.

Table 2.—Geometric standards [1] for new construction of two- and four-lane primary rural State highways

Design features	Two-lane highways with future average daily traffic volumes of—												Two-lane highways on Interstate System, and located in—			Four-lane highways located in—		
	Less than 500 ADT, and located in—			500–1,800 ADT, and located in—			1,800–3,000 ADT, and located in—			Over 3,000 ADT, and located in—								
	Valley terrain	Rolliug terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain [4]	Valley terrain	Rolling terrain	Moun-tainous terrain
Design speed—m. p. h.	50	40	35	60	50	40	70	60	50	70	60	50	70	60	60	70	60	60
Operating speed—m. p. h.	40–45	35–40	30–35	45–50	40–45	35–40	45–50	45–50	40–45	50–55	45–50	40–45	50–55	45–50	45–50	50–55	45–50	45–50
Maximum curvature—degrees	9	14	20	6	9	14	4	6	8	3	5	7	3	5	5	3	5	5
Stopping sight distance—feet	350	275	225	525	350	275	700	525	350	700	525	350	700	525	525	700	525	525
Lane width—do	10	10	10	[2] 11	[2] 11	[2] 11	12	12	12	12	12	12	12	12	12	[3] 12	[3] 12	[3] 12
Shoulder width:																		
In cut—do	5	3	3	6	4	4	8	6	4	8	8	4	8	8	4	8	8	4
On fill—do	7	5	5	8	8	7	10	8	8	10	10	8	10	10	8	10	10	8
Percentage of passing sight distance available:																		
1,500 feet—percent	[4]	[4]	[4]	10	5	———	10	5	———	10	5	———	10	5	———	[5]	[5]	[5]
1,000 feet—do	[4]	[4]	[4]	———	———	5	———	———	[6] 5	———	———	[6] 5	———	———	[6] 5	[5]	[5]	[5]
Grade [7]—do	5	7	9	4	5	7	7	7	7	7	7	7	7	7	7	7	7	7
Right-of-way width [8]—feet	60	60	60	80	80	80	100	100	100	120	120	120	120	120	120	200	200	200
Surface type	Low	Low	Low	Medium	Medium	Medium	High	High	High	High	High	High	High	High	High	High	High	High
Bridges: [9]																		
Loading [10]	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16
Minimum vertical clearance—do	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.
[2] On new construction or reconstruction, 12-foot lanes should be used with volumes above 1,200 vehicles per day when truck traffic is more than 5 percent.
[3] All 4-lane highways shall be divided. Whenever possible the median shall be at least 20 feet wide, and in no case less thau the 4-foot barrier type.
[4] Percentage feasible.
[5] Not applicable.
[6] Five percent or as required for capacity.
[7] For 2-lane highways carrying less than 500 vehicles per day, add 2 percent for grades under 750 feet long; for 2-lane highways carrying from 500 to 1,800 vehicles per day, add 1 percent for grades under 750 feet long; for 2-lane highways carrying 1,800 and more vehicles per day and Interstate System routes, the 7-percent grade is maximum unless percentage and length of grade are determined by capacity calculation.
[8] Control of access shall be provided on all new locations on the Interstate System and on other primary highways carrying large volumes of traffic.
[9] For bridges carrying less than 3,000 vehicles per day, the width of the structure, if less than 50 feet long, shall be the full roadway width; if the length of the structure is greater than 50 feet, the width shall be that of the approach pavement plus 4 feet and the width of the median in the case of 4-lane divided highway.
[10] For highways carrying less than 3,000 vehicles per day, the loading shall be H15–S12 or oue H20 truck, whichever produces the greater stress; for highways carrying 3,000 or more vehicles per day and a volume of heavy truck traffic comparable to Interstate System routes, the loading shall be H20–S16.

Table 3.—Tolerable conditions [1] for two- and four-lane primary rural State highways

Design features	Two-lane highways with future average daily traffic volumes of—												Two-lane highways on Interstate System, and located in—			Four-lane highways located in—		
	Less than 500 ADT, and located in—			500–1,800 ADT, and located iu—			1,800–3,000 ADT, and located in—			Over 3,000 ADT, and located in—								
	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain	Valley terrain	Rolling terrain	Moun-tainous terrain
Design speed—m. p. h.	40	35	30	50	40	35	60	50	40	60	50	40	60	50	45	60	50	45
Operating speed—m. p. h.	35–40	30–35	25–30	40–45	35–40	30–35	45–50	40–45	35–40	45–50	40–45	35–40	45–50	40–45	40–45	45–50	40–45	40–45
Maximum curvature—degrees	14	20	25	9	14	20	6	9	14	5	7	11	5	7	9	5	7	9
Stopping sight distance—ft.	275	225	200	350	275	225	475	350	275	475	350	275	475	350	300	475	350	300
Lane width—ft.	9	9	9	10	[2] 10	[2] 10	10	10	10	10	10	10	10	10	10	10	10	10
Shoulder width:																		
In cut—ft.	3	3	2	6	3	3	6	4	4	6	4	4	6	4	4	6	4	4
On fill—ft.	3	3	3	6	4	4	8	6	4	8	6	4	8	6	4	8	6	4
Percentage of passing sight distance available:																		
1,500 feet—pct.	[3]	[3]	[3]	[3]	[3]	[3]	10	5	———	10	5	———	10	5	———	[4]	[4]	[4]
1,000 feet—pct.	[3]	[3]	[3]	[3]	[3]	[3]	———	———	[5] 5	———	———	[5] 5	———	———	[5] 5	[4]	[4]	[4]
Grade [6]—pct.	7	8	10	5	7	8	8	8	8	7	7	7	7	7	7	7	7	7
Surface type [7]	Low	Low	Low	Low	Low	Low	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium
Bridges: [8]																		
Loading	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15
Minimum vertical clearance—ft.	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.
[2] Lane widths of 9 feet may be accepted as tolerable for volumes under 800 vehicles per day with a small percentage of trucks.
[3] Not required. [4] Not applicable. [5] 5 perceut or as required for capacity.
[6] For two-lane highways carrying less than 500 vehicles per day, add 1 percent for grades under 1,000 feet long; for two-lane highways carrying from 500 to 1,800 vehicles per day, add 1 percent for grades under 750 feet long; for two-lane highways carrying 1,800 or more vehicles per day and Interstate System routes, the 7-percent grade is maximum unless percentage and length of grade are determined by capacity calculation.
[7] For highways carrying 500 or more vehicles per day, a good road surface condition is required; control of access is not required.
[8] For bridges carrying less than 500 vehicles per day, the width of the structure should not be less than the width of the approach pavement; for bridges carrying 500 or more vehicles per day, the width of the structure should not be less than the width of the approach pavement plus 2 feet.

Table 4.—Average daily capacities of two-lane highways with 12-foot traffic lanes and constructed to a given design speed [1]

Percentage of highway with passing sight distance [2] of—		Valley or flat terrain with operating speed, 50–55 m. p. h.; design speed, 70 m. p. h.	Rolling terrain with operating speed, 45–50 m. p. h.; design speed, 60 m. p. h.	Mountainous terrain	
1,500 feet	1,000 feet			Interstate System with operating speed 45–50 m. p. h.; design speed, 60 m. p. h.	Other than Interstate System with operating speed, 40–45 m. p. h.; design speed, 60 m. p. h.
		V. p. d.	*V. p. d.*	*V. p. d.*	*V. p. d.*
100	100	4,850	6,500	5,500	6,550
80	90	4,450	5,850	5,000	6,000
60	80	3,950	5,050	4,300	5,300
40	70	3,350	4,200	3,600	4,600
20	60	2,400	3,450	2,900	3,850
0	50	1,300	2,600	2,200	3,050

[1] Based on 5-percent truck traffic during 30th highest hour (12 percent of ADT).
[2] Percentage of 1,500-foot passing sight distance is used for all operating speeds except those below 45 miles per hour. The 1,000-foot values are applicable to all operating speeds.

greed upon, it was relatively easy to establish the design requirements for new construction and tolerable conditions, as shown n tables 2 and 3, from the information now ontained in the AASHO policy on *Geometric Design of Rural Highways* (*1*) and to prepare able 4 from the results of traffic operation and apacity studies conducted during the past everal years.

Table 4 shows the average daily traffic olumes that can be accommodated by a wo-lane highway constructed to a given esign speed with various percentages of the ighway having sight distances in excess of ,500 and 1,000 feet. The values in this table re for the following average conditions pplicable in West Virginia:

1. The 30th highest hourly volume during he year is 12 percent of average daily traffic or that year.

2. During the 30th highest hourly volume f a year, trucks with dual tires account for percent of the traffic.

3. In a capacity sense, the average dual-tired truck is equivalent to 2 passenger cars in valley or level terrain, to 4 passenger cars in rolling terrain, and to 8 passenger cars in mountainous terrain.

For highways where these average conditions do not exist or are not expected to be present during the year for which the highway is designed, appropriate corrections must be made in the capacities by the application of factors similar to those included in the discussion on capacities of existing highways.

Design Speeds of Existing Highways

If the AASHO definition of design speed were applied to existing highways with profiles and alinements that were constructed prior to the time that this term came into common usage, it would be found that in many cases the average running speed of traffic would be several miles per hour above the design speed. Likewise, recent studies have shown that highways constructed to modern standards may provide radically different operating conditions even though their traffic volumes and design speeds are identical. Average speeds, for example, will be much higher on a highway with few 5-degree curves and a lot of tangent alinement than on a highway with many 5-degree curves and little tangent alinement. This is because above-minimum design values are utilized where feasible and drivers do vary their speeds to a considerable extent with the immediate geometric conditions rather than adopting one uniform speed for the entire length of a highway.

Conversely, for a given operating speed, a highway with few curves and mostly tangent alinement will accommodate higher volumes of traffic than a similar highway with many curves of the same degree and less tangent alinement. In relating the operating speed of a highway to its capacity, it is therefore necessary to determine the average highway speed, especially for existing highways.

The introduction of the term "average highway speed," which is in effect the average maximum safe speed or the operating speed for a passenger car over a section of highway during extremely low traffic densities, is an approach which has not previously been employed in relating alinement and profile to capacities. It is, however, an approach which must be employed to obtain reasonable accuracy in capacity determinations, especially for existing highways.

The average highway speed of an existing highway may be determined by weighting the possible speeds of traffic on the individual sections during low traffic flows by the length of the sections. The possible speeds for various horizontal curves and stopping sight distance conditions may be determined by use of the AASHO tables (*1*) relating these features to the design speed.

When preparing plans for a highway, the designer should base the geometric features on an assumed design speed over a substantial length of highway to obtain a balanced design. Invariably there are sections where the designer utilizes values that are adequate for higher speeds than the design speed which he has assumed. The lower the design speed, the greater is the likelihood of the occurrence of such sections. As a result, the high-speed driver can travel over the section during low traffic densities at an average speed which exceeds the assumed design speed. This speed is the average highway speed and is equivalent to the low-volume operating speed.

Figures 1 and 2 show how the operating speeds on a two-lane highway vary with the average highway speeds, the percentage of highways having 1,500-foot passing sight distance, and the traffic volumes. The average daily traffic volumes in these charts are based on highways located in essentially level terrain, with 12-foot traffic lanes, carrying 5-percent dual-tired vehicles with a passenger-car equivalent of 2, and a 30th highest hourly volume during the year of 12 percent of the average daily traffic. The charts were prepared for the Tennessee Programing Study from the information contained in table 5

Table 5.—Average daily capacities of two-lane highways located on level terrain and carrying 5-percent truck traffic during the 30th highest hour [1]

Operating speed	Percentage of highway with passing sight distance of—		Average highway speed of—					
	1,500 feet	800 to 1,000 feet	70 m. p. h.	60 m. p. h.	55 m. p. h.	50 m. p. h.	45 m. p. h.	40 m p. h.
M. p. h.			*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*
50–55	100	100	4,850	4,750	4,300	.....	.....	.....
50–55	80	92	4,450	4,150	3,750	.....	.....	.....
50–55	60	84	3,950	3,450	3,000	.....	.....	.....
50–55	40	76	3,350	2,700	2,250	.....	.....	.....
50–55	20	68	2,400	1,800	1,350	.....	.....	.....
50–55	0	60	1,300	900	550	.....	.....	.....
45–50	100	100	7,100	7,100	6,600	5,650	.....	.....
45–50	80	90	6,800	6,400	5,750	5,050	.....	.....
45–50	60	80	6,400	5,550	4,800	4,000	.....	.....
45–50	40	70	5,750	4,600	3,900	2,800	.....	.....
45–50	20	60	4,900	3,750	2,800	2,000	.....	.....
45–50	0	50	3,800	2,800	2,000	1,250	.....	.....
40–45	100	100	8,450	8,450	8,050	7,450	6,550	.....
40–45	80	87	8,250	7,700	7,300	6,700	5,800	.....
40–45	60	76	7,900	6,800	6,450	5,700	4,850	.....
40–45	40	64	7,350	5,900	5,300	4,600	3,700	.....
40–45	20	52	6,650	4,950	4,100	3,200	2,200	.....
40–45	0	40	5,850	3,950	2,700	1,950	1,250	.....
35–40	100	100	9,900	9,900	9,900	9,600	9,100	7,900
35–40	80	85	9,750	9,350	9,000	8,750	8,200	7,150
35–40	60	72	9,400	8,650	8,100	7,950	7,250	6,000
35–40	40	58	8,950	8,000	7,150	6,950	6,150	4,700
35–40	20	44	8,400	7,350	6,300	5,850	4,500	3,100
35–40	0	30	7,650	6,600	5,350	3,850	2,450	1,500

[1] A truck factor of 2.0 for West Virginia; the normal factor is 2.5. The 30th highest hourly volume is 12 percent of ADT.

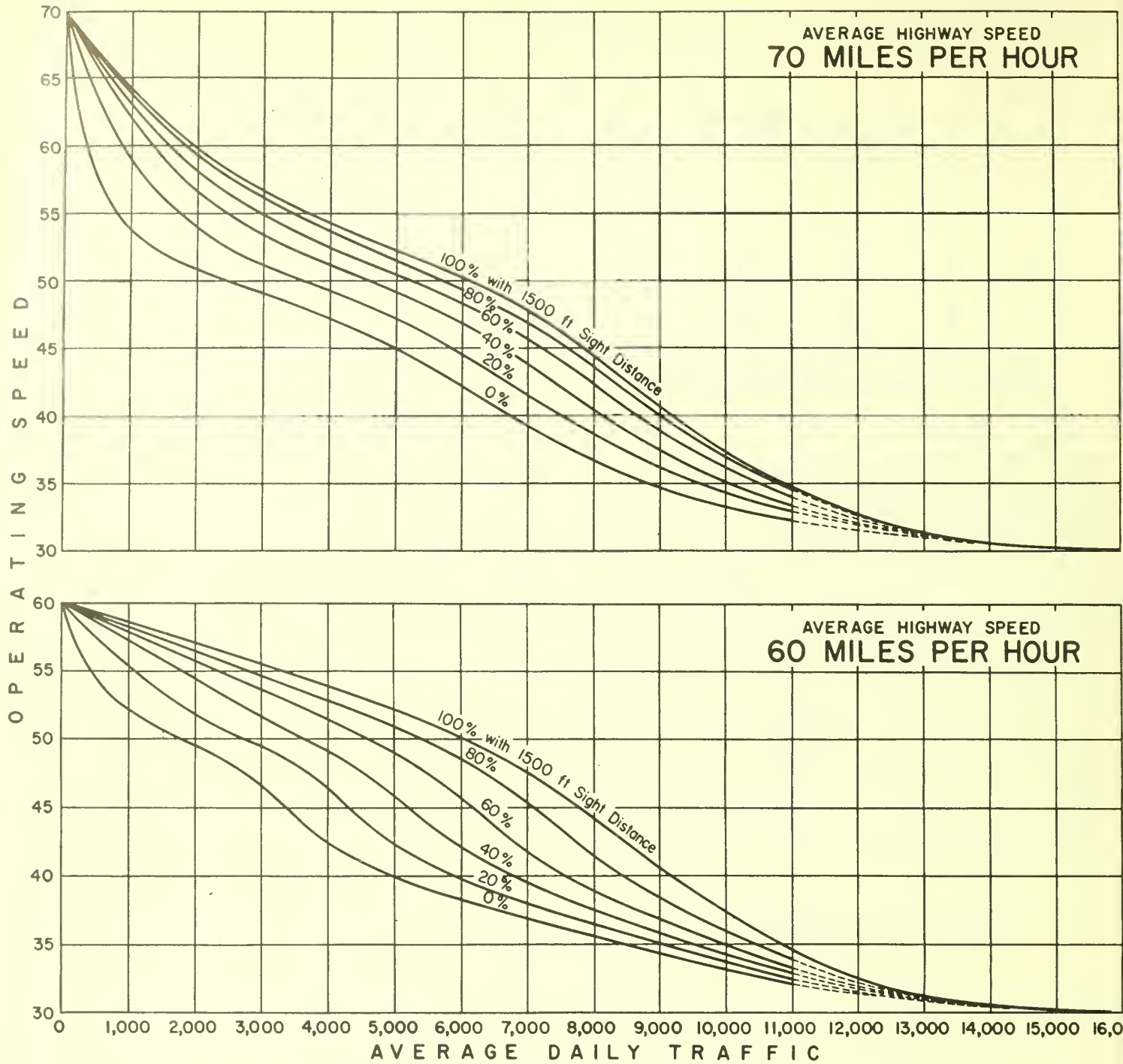

Figure 1.—Effect of traffic volumes and available passing sight distances on operating speeds with average highway speeds of 60 and miles per hour.

which was prepared for the 1953–54 West Virginia Needs Study. Table 5 in turn was prepared from the results of extensive highway capacity studies conducted by the Bureau of Public Roads in cooperation with the various State highway departments and includes the results reported in the *Highway Capacity Manual* (*2*) supplemented by more recent investigations.

In figures 1 and 2 there are curves representing roadways with sight distances that are continuously in excess of 1,500 feet to those which have no sections with 1,500-foot sight distance. The relation between operating speed and traffic volume as shown by the curves is applicable, however, only when the percentage of the highway not having a 1,500-foot sight distance is fairly evenly distributed between the limits of 1,500 feet and the stopping sight distance for the design speed. This is the more usual condition.

It must be pointed out that most of the data on which figures 1 and 2 are based were obtained by studies conducted during traffic volumes within the lower three-quarters of the range (below 12,000 ADT). Studies conducted on two-lane highways during capacity volumes represent principally level tangent sections well removed from sharp horizontal or vertical curves. For this reason, all except the top curves (100 percent with with 1,500-foot sight distance) are shown as broken lines for traffic volumes above 11,0 vehicles per day. There is still considerab question as to whether all the curves for t same average highway speed meet at a co mon point on the right, or whether the possib capacity and the speed at this capacity a slightly lower for the highways with t poorer alinement than for those with a co tinuous sight distance in excess of 1,500 fe This, however, is not too important a co sideration because the practical capacities two-lane highways are well within the ran for which reliable data are available.

The charts may be used either to determi the operating speed for a given traffic volum or the traffic volume which the highway wi

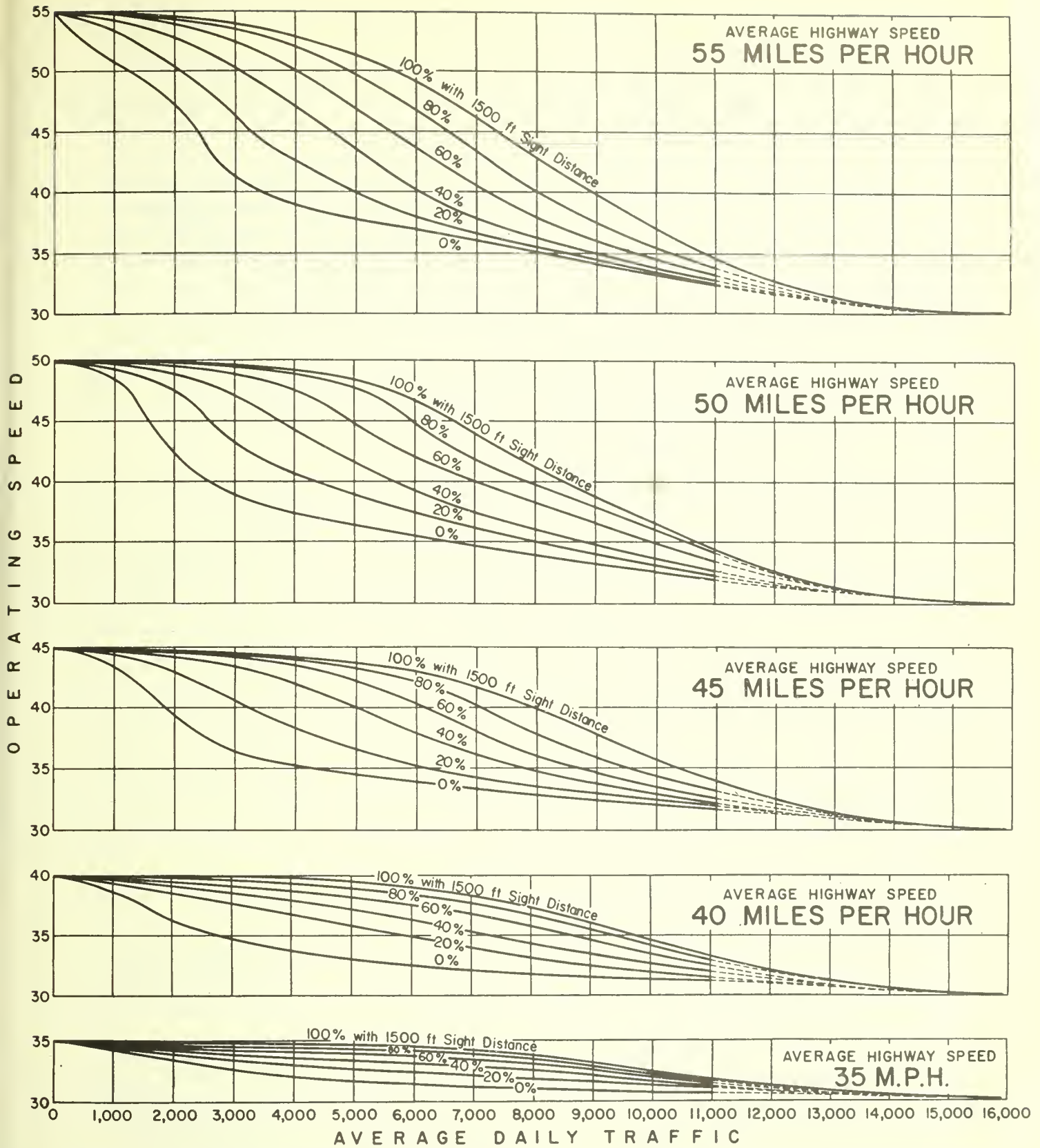

Figure 2.—Effect of traffic volumes and available passing sight distances on operating speeds with average highway speeds of 35, 40, 45, 50, and 55 miles per hour.

ccommodate at a given operating speed. When it is desired to determine the capacity t a given operating speed for lane widths ther than 12 feet, for 30th highest hourly actors other than 12 percent, for truck per-entages other than 5 percent, or for truck quivalents other than 2, the following procedure must be applied to adjust the capacity volumes to the prevailing or estimated future conditions:

1. For 11-foot lanes multiply the volumes by 0.86, and for 10-foot lanes multiply by 0.77.

2. When the 30th-highest-hour factor is other than 12 percent, multiply the volumes by 12/actual percentage.

3. When there is other than 5 percent trucks during the peak hour or the truck equivalent is greater than 2, as it will be on grades and in rolling or mountainous terrain, multiply the volumes by—

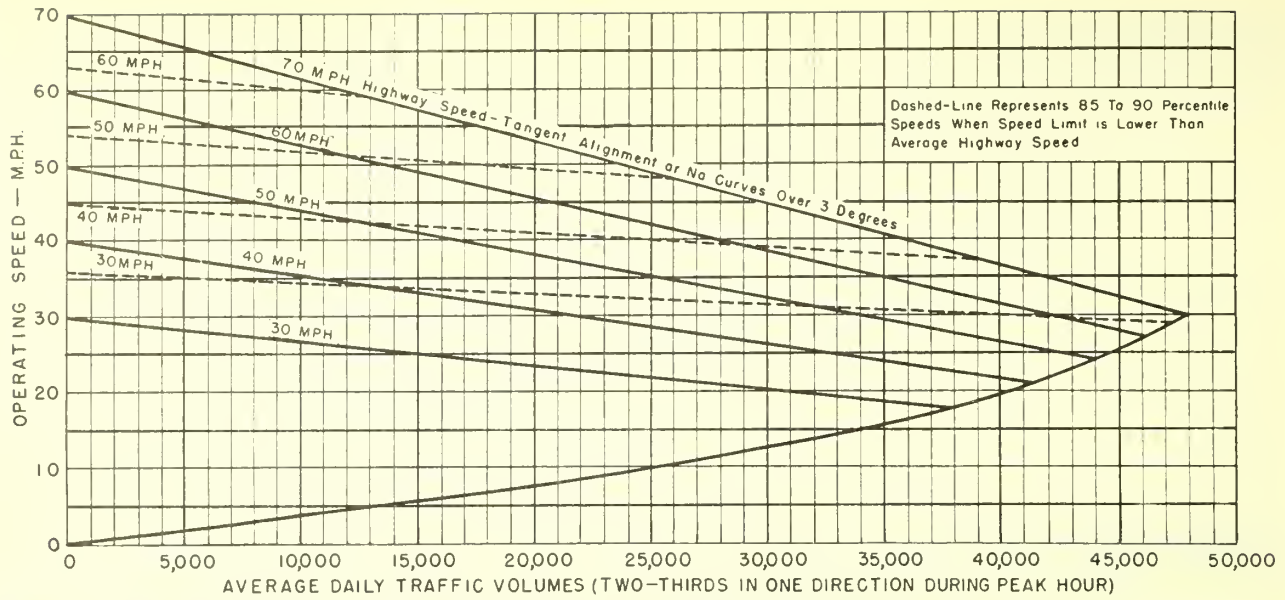
OPERATING SPEED — M.P.H.
AVERAGE DAILY TRAFFIC VOLUMES (TWO-THIRDS IN ONE DIRECTION DURING PEAK HOUR)
60 MPH
70 M.P.H. Highway Speed - Tangent Alignment or No Curves Over 3 Degrees
50 MPH
60 MPH
50 MPH
40 MPH
30 MPH
40 MPH
30 MPH
Dashed-Line Represents 85 To 90 Percentile Speeds When Speed Limit is Lower Than Average Highway Speed

Figure 3.—Operating speeds on four-lane highways in the direction of heavier travel for various average highway speeds.

$$\frac{105}{100-P+PT}$$

Where:

P = The percentage of trucks.
T = The truck equivalent in terms of passenger cars.

The operating speed for a given traffic volume, when conditions other than those used for figures 1 and 2 are applicable, may be determined by employing the reciprocal of the correction factors shown in items 1 through 3 to the given traffic volume before entering in the chart.

Tables A–H are included at the close of this article on pages 38–39 for the conditions most prevalent on two-lane highways in West Virginia. The number of charts or tables that can be prepared for other combinations of the many variable conditions is almost unlimited. A similar set of tables may be prepared for the conditions prevailing within any State or area.

Operating and Average Speeds on Four-Lane Divided Highways

Figure 3 shows the relation between operating speeds, average highway speeds, and traffic volumes on four-lane divided rural highways free from the influence of intersections. The lowest curve represents the minimum speed at which traffic must flow to attain a given traffic volume. For example, traffic must be traveling at least 10 miles per hour for a four-lane highway to accommodate the 30th highest hourly volume when the average daily traffic is 25,000 vehicles.

The other solid lines in figure 3 represent the normal operating speeds during various traffic volumes for different average highway speeds. Any point representing the speed-volume relation must fall between the lower curve and the line representing the average highway speed.

The dashed-lines show the effect of an enforced speed limit on the speed-volume relation. A speed limit has an effect on the operating speed only when it is lower than the highway speed. Also, it has an effect only when the traffic volume is below the volume at which the dashed-line (speed limit) intersects the solid line corresponding to the highway speed. At higher volumes, the solid lines show the normal speed-volume relation, since at these volumes the speeds are governed by the traffic density rather than by the speed limits.

Figure 4 is similar to figure 3 except that average speed rather than operating speed is related to the traffic volume. Figure 3 also shows the daily volumes based on a 30th-highest-hour factor of 12 percent and includes

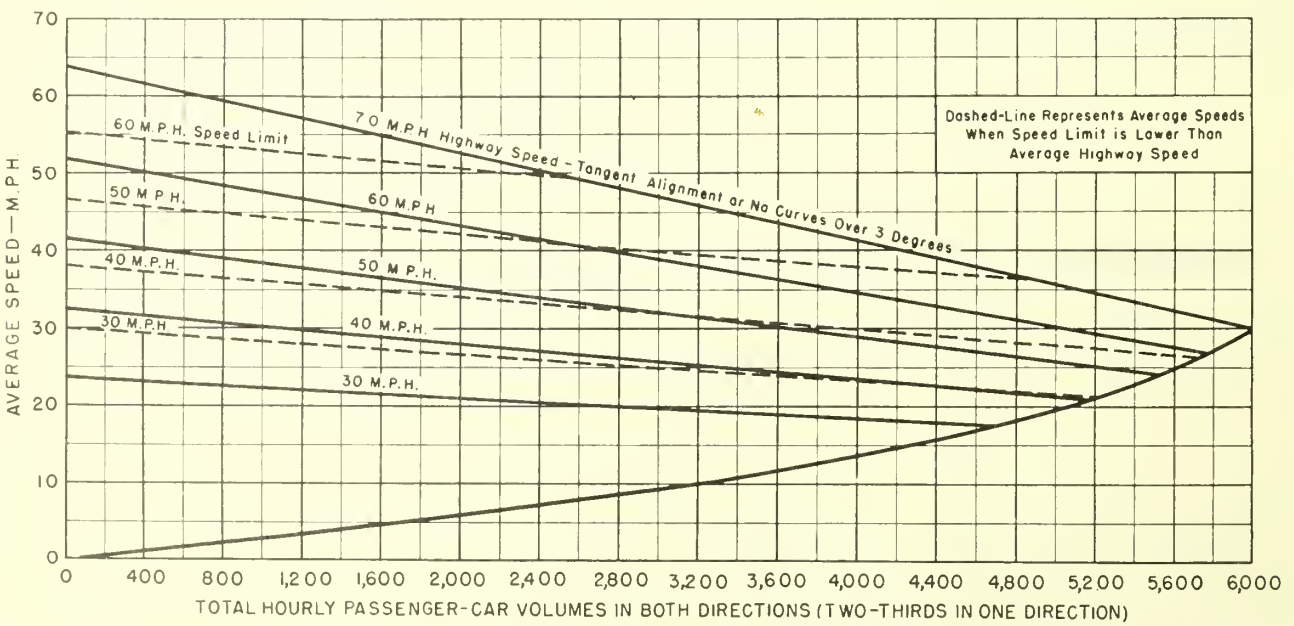
AVERAGE SPEED — M.P.H.
TOTAL HOURLY PASSENGER-CAR VOLUMES IN BOTH DIRECTIONS (TWO-THIRDS IN ONE DIRECTION)
60 M.P.H. Speed Limit
70 M.P.H Highway Speed - Tangent Alignment or No Curves Over 3 Degrees
50 M.P.H.
60 M.P.H
40 M.P.H.
50 M.P.H.
30 M.P.H
40 M.P.H.
30 M.P.H.
Dashed-Line Represents Average Speeds When Speed Limit is Lower Than Average Highway Speed

Figure 4.—Average speeds on four-lane highways in the direction of heavier travel for various average highway speeds.

5 percent trucks with a passenger-car equivalent of 2, whereas figure 4 shows hourly volumes and includes no trucks.

These two charts represent average conditions found on modern highways throughout the United States. In some areas, such as the central States where the terrain is level and speeds are higher than for the country as a whole, the speeds as shown by these charts will be somewhat low especially for the low traffic volumes. For certain other areas they may be high, but in general any difference will not be great and the relative speeds for the different conditions will be accurate.

The traffic volumes or capacities at a given operating speed or at a given average speed are shown in terms of numbers of vehicles in two 12-foot lanes for the one direction of travel. Daily and hourly volumes or capacities for various percentages of trucks and a range of truck factors may be determined by standard procedures.

The results for multilane highways as shown by figures 3 and 4 explain to a large extent the many variations in the speed-volume relation found by other investigators. Sometimes they have found that an increase in the traffic volume or density results in only a very slight or no drop in speeds. This would be the case as shown by the dashed-lines in figures 3 and 4 when a speed limit or factors other than the traffic density are exerting a controlling influence on vehicle speeds.

The results of still other investigators show a curvilinear relation with the speeds dropping at an increasing rate as the traffic density increases. This would occur as the traffic volumes exceed the range within which the speed limits are effective and especially when the volumes approach possible capacities. At volumes approaching possible capacities on multilane facilities (above 1,500 vehicles per hour per lane), the safety factor for capacity, as indicated by the distance between the upper and lower curves of figures 3 and 4, decreases rapidly with the result that a slow driver or some other minor condition interrupting the normal flow of traffic can cause a sudden slowdown of all vehicles with speeds decreasing from a point on one of the higher curves of figures 3 and 4 to a point on the bottom curve, or to any intermediate point. The closer the possible capacity is approached, the greater is the possibility of such an occurrence.

The most baffling results obtained from speed-volume investigations are those which show an increase in speed with an increase in volume. Generally this occurs when a study is started during off-peak hours with light traffic and is continued through the peak or rush-hour volumes in the afternoon. As the traffic volume increases, the percentage of repeat drivers in a hurry to get home increases with the result that speeds show little or no decline and oftentimes increase temporarily with the traffic volume. When capacity volumes are reached or closely approached there is then an abnormal decrease in speeds producing the curvilinear relation between speed and traffic volume. Studies of this type do not show the true effect of increased volume or speeds, since there is a marked change in the character of traffic from off-peak to peak periods. The true effect of volume on speeds as shown in figures 3 and 4 can be obtained by simultaneous studies at different points where the geometric features of the highway are identical but the traffic volumes are different.

Information Needed for Capacity Analysis

An engineering analysis of the ability of a highway to accommodate present or estimated future traffic volumes, in accordance with prescribed standards of service in terms of operating speeds, requires the following information:

1. The type of terrain through which the highway is located.
2. The average highway speed and the frequency of occurrence of sharp curves that cause abnormally low speeds.
3. The percentage of the highway on which the passing sight distance exceeds 1,500 feet. On highways for which an operating speed of 40 miles per hour or less has been specified, the percentage of highway with an 800- to 1,000-foot sight distance is required whenever there is a low percentage of the 1,500-foot sight distance.
4. The average truck factor and the truck factor on all long or steep grades.
5. Cross-section items such as shoulder and surface type, width, and condition.

These five items were determined for all highways in West Virginia expected to carry annual volumes in excess of 1,800 vehicles per day within the next 20 years.

Type of terrain

Generally the alinement of an existing highway will be an indication of the surrounding terrain. Whether standards for level, rolling, or mountainous terrain should be applied to an existing road is largely a matter of engineering judgment. Just because the existing highway has many sharp curves and steep grades, however, does not necessarily mean that a much better alinement and profile could not be obtained in the same general vicinity at a reasonable cost with modern equipment and methods. A large part of West Virginia has terrain through which it is extremely difficult and costly to build high-speed highways of modern design.

Average highway speed

The average highway speed of each section of highway was determined by driving a passenger car over the highway at the maximum safe speed during extremely low traffic volumes to obtain a profile of the speed based on the geometric features of the highway. The safe speed was governed by sight distance, curvature, and possible marginal interferences. All speed zones and speed limits were observed. Long tangent sections of highway were recorded as having a 60-miles-per-hour highway speed even though the test car was not necessarily operated at that speed. Such sections are, however, comparatively rare in West Virginia.

This method of determining the average highway speed and obtaining a log of the sharp curves and other speed restrictions was employed because sufficiently detailed information was not available from any other source. Furthermore, this method as it was employed was sufficiently accurate and probably resulted in a more realistic appraisal than could have been obtained from detailed plans had they been available.

Passing sight distance

A second car with an accurate odometer was driven over each highway at a slow speed (about 30 m.p.h.) to determine the length and location of all sections with sight distances in excess of 1,000 feet and 1,500 feet, in lieu of more accurate and detailed sight distance information. The driver informed the passenger, who acted as the recorder, each time there was a change in the sight distance from some value below 1,000 feet or 1,500 feet to a value above 1,000 or 1,500 feet. He also informed the recorder each time the sight distance again became less than either of these values.

The recorder noted the odometer readings at these locations and at control points such as crossroads, city limits, and major structures. It was possible to check the accuracy of the driver's estimate by this procedure as each reading was recorded so that a sufficiently accurate estimate was obtained of the percentage of the highway with a sight distance in excess of 1,000 feet and the percentage in excess of 1,500 feet.

Average truck factor

Commercial vehicles with dual tires reduce the capacity of a highway in terms of vehicles per hour. In level terrain where commercial vehicles can maintain speeds that equal or approach the speeds of passenger cars, it has been found that the average dual-tired vehicle is equivalent, in a capacity sense, to 2 passenger cars on multilane highways and to 2.5 passenger cars on two-lane highways. The number of passenger cars that each dual-tired vehicle represents is termed the "truck equivalent" or the "truck factor."

The results of highway capacity studies have shown that the truck equivalent on long or steep grades increases with an increase in the difference between the normal speeds of passenger cars and the speeds of trucks. They have also shown that the truck equivalent changes very little, if at all, with a change in the percentage of trucks in the total traffic stream.[3]

Truck equivalents are normally determined by obtaining detailed information on the speeds and headways of vehicles during various traffic volumes on highways with different alinements and profiles. An average truck factor is obtained for dual-tired vehicles under each condition. If the study is of sufficient magnitude, it is possible to obtain

[3] Studies have not been conducted at locations with more than 20 percent dual-tired trucks and have been confined principally to locations with less than 10 percent of these vehicles during the periods of peak flow. Further studies may indicate that for certain conditions the truck factor does change with a change in the percentage of trucks, but as yet there is no evidence to indicate whether it increases or decreases with an increase in the number or percentage of trucks.

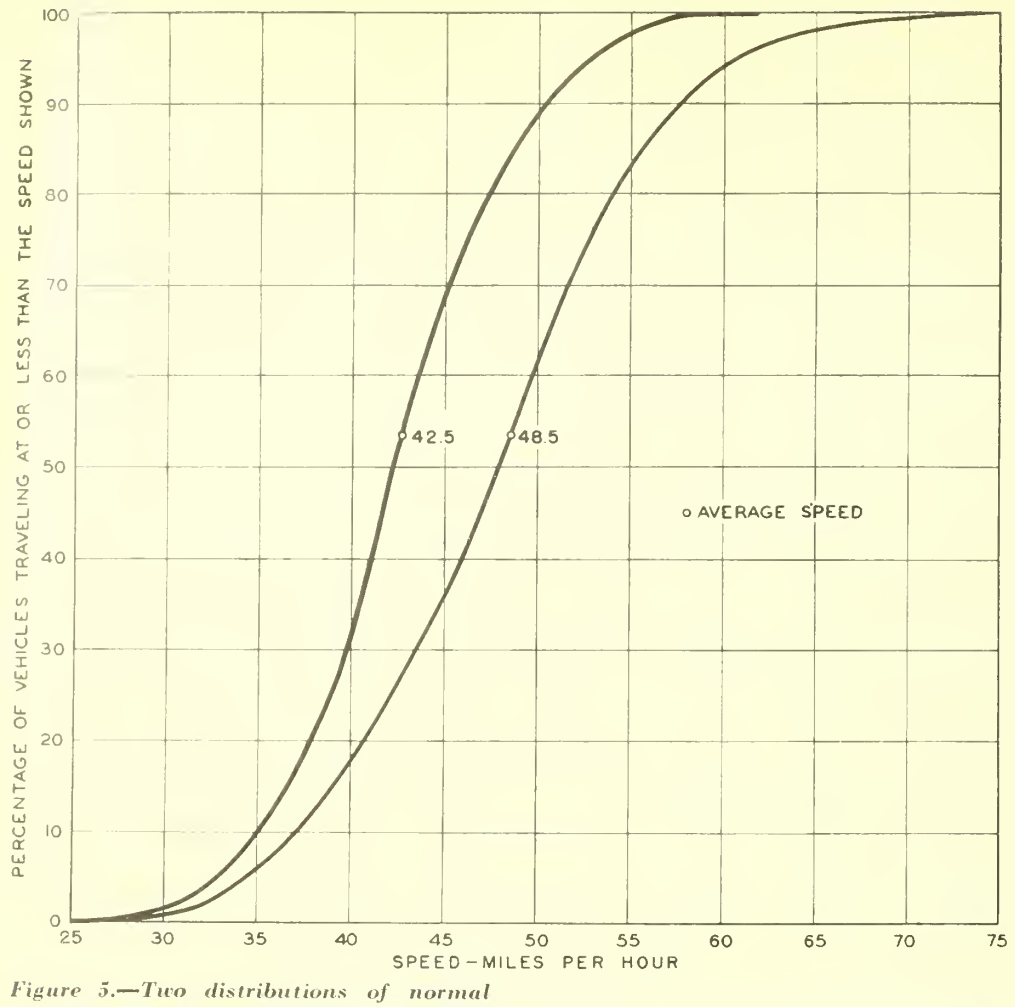

Figure 5.—Two distributions of normal passenger-car speeds used in determining truck factors (passenger-car equivalents).

a truck factor for each type of dual-tired vehicle classified by speed groups.

The results of these studies have shown that truck factors can also be calculated with a high degree of accuracy from the separate speed distributions of passenger cars and trucks recorded during light volumes when vehicles can travel at their normal speeds. The criterion used is the relative number of passings that would be performed per mile of highway if each vehicle continued at its normal speed for the conditions under consideration. That the results from such an analysis agree with those obtained by the more painstaking methods is not surprising. It is the difference between truck speeds and passenger-car speeds on grades that causes trucks to reduce the capacity of a highway. The greater the speed difference, the greater is the reduction in capacity with a corresponding increase in the truck factor.

Table 6 shows how the truck factor varies with the truck speed for two different passenger-car speed distributions as shown in figure 5. The higher the passenger-car speeds, the higher are the truck equivalents. The factors in the right-hand column of table 6 are the rounded values used for the West Virginia study and from which figure 6 was plotted. The truck equivalent can be determined for any dual-tired vehicle by knowing its average speed under any highway condition such as a steep or long grade. The average truck factor can also be determined for any location or section of highway by knowing the average speed for all trucks if the passenger-car speeds are within the limits of those shown in figure 5. In this case, there will be a slight error if there is a wide range in the truck speeds because the curve of figure 6 is not a straight line. The error will be slight, however, for most conditions.

Table 6.—Truck factor for various truck speeds as related to normal passenger-car speeds

Truck speed (miles per hour)	Truck factor—passenger-car equivalents		
	For average passenger-car speed of 48.5 m. p. h.[1]	For average passenger-car speed of 42.5 m. p. h.[1]	Adopted for use in West Virginia study
40	1.8	1.5	2
35	3.0	2.7	3
30	5.0	4.9	5
25	8.6	7.6	8
20	13.9	11.7	13
15	22.9	18.7	20
10	40.5	32.5	35
5	94.5	75.0	80

[1] Distributions of passenger-car speeds are shown in figure 5.

Control Truck Used for Obtaining Average Truck Speed

In flat or rolling terrain it is possible to conduct sufficient speed studies to determine the speeds of trucks for the typical and unusual profiles that are encountered on a

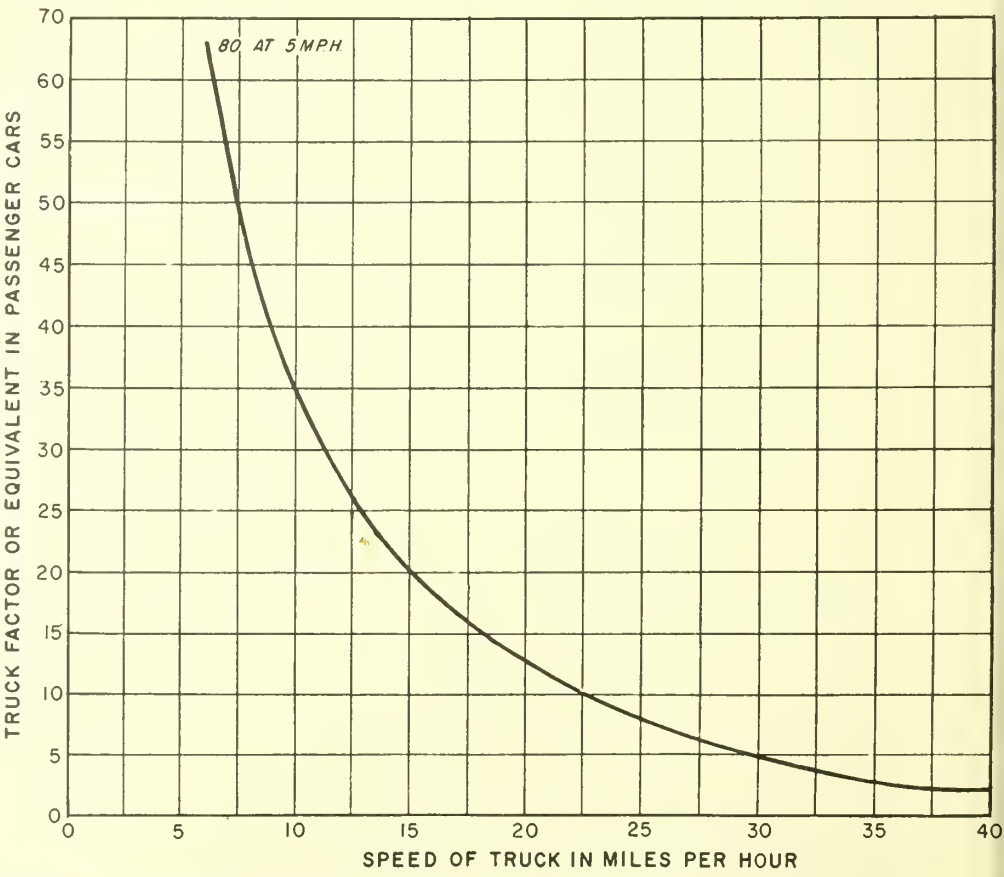

Figure 6.—Truck factors (passenger-car equivalents) for average truck speeds ranging from 5 to 40 miles per hour.

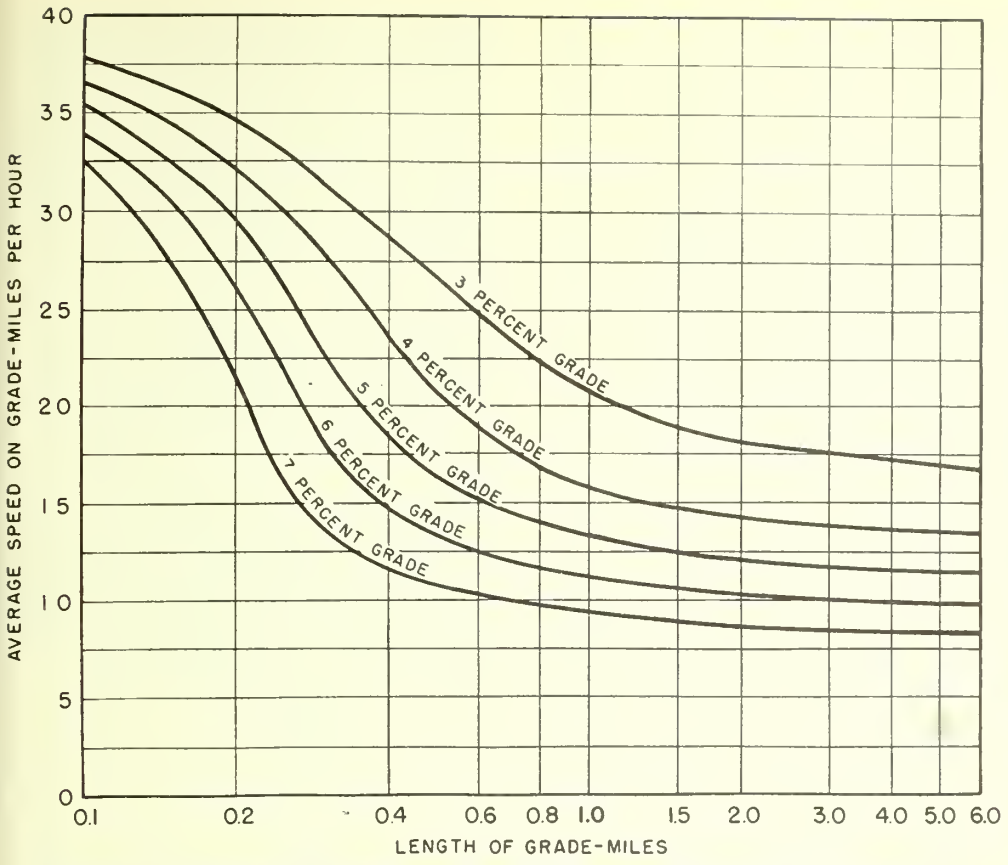

Figure 7.—Average speeds of control truck on grades ranging from 3 to 7 percent.

highway system. In mountainous terrain, however, this approaches an impossible task. This is especially true for West Virginia. A unique method was therefore employed to obtain the average truck factor for each section of highway and for each grade or combination of grades on all roads in West Virginia likely to carry more than 1,800 vehicles per day during the next 20 years.

The method involved the selection of a typical truck with a typical load. This truck was driven over the highway system at its maximum safe speed consistent with normal truck operation to obtain a continuous speed profile. The speed of the truck and its odometer reading were recorded at the bottom and top of each grade, at crossroads or other control points, each time the gears were shifted, and each time there was a change of 5 miles per hour in the speed of the truck. When the truck reached a crawl speed on long grades, the crawl speed was recorded to the nearest mile per hour. The truck was operated in both directions on the more important roads to get a speed profile for each direction of travel.

The control truck and its load were selected to obtain a weight-power ratio of 325 pounds per horsepower so that its effect on highway capacity would be the same as the average dual-tired vehicle. Its gross load was 40,000 pounds which is considerably lighter than the heaviest group of vehicles recorded during recent loadometer surveys, but also heavier than the average dual-tired vehicle including those with and without payloads. Since the curve in figure 6 is not a straight line, the possible speed of the control truck on an upgrade was purposely recorded somewhat lower than the average for all dual-tired trucks on the same grade. This was necessary so that the truck factor obtained for the speed of the control truck from figure 6 would equal the average factor for all trucks.

As an example, the average truck factor for speeds of 35 and 15 miles per hour is 11.5 or $(3+20)\div 2$. A truck factor of 11.5 is represented by a speed of 21 miles per hour rather than 25 miles per hour—the average of 35 and 15.

Soon after placing the control truck in operation, its speeds on hills with known gradients were checked with the performance curves for vehicles under controlled test conditions and found to be in agreement. Trial runs on the same grade were also remarkably consistent.

Speed data for trucks on grades, recorded at spot locations and also over the entire length of long grades by the stopwatch method, showed that the average truck factor was somewhat lower than the truck factor obtained by using the speed of the control truck. The difference varied from 10 to 20 percent. Since this was on the conservative side and would make a difference of less than 5 percent when used for estimating the capacities of existing roads, no adjustment or correction was made. Had it been desired to more accurately duplicate the average performance of present-day commercial vehicles as found in West Virginia, the load on the control truck should have been reduced about 5,000 pounds.

The average speeds of the control truck on 3 to 7 percent uniform grades up to 6 miles long are shown in figure 7 and table 7. Figure 8 shows the speed of the truck at any point on these grades. The speeds as shown by the solid lines are based on the assumption that the truck enters the grade at 41 miles per hour.

These curves may also be used to determine the speed reduction due to any length and steepness of grade for other approach speeds. For example, if the approach speed is 40 miles per hour (initial distance 85 feet), the speed at the top of a 4-percent grade 1,000 feet long will be 26 miles per hour (final distance 1,085 feet). Similarly, if this same grade is approached at a speed of 30 miles per hour, the speed at the top will be 17 miles per hour.

The dashed-curves emanating from 9 miles per hour show the maximum performance of vehicles when the approach speed is so low that the vehicle must accelerate to eventually reach the sustained speed. These curves show that it takes exceedingly long distances to accelerate on grades when the approach speed is below that of the sustained speed. To change the speed on a 2-percent grade from 20 miles per hour to the sustained speed of 21.5 miles per hour, an increase of only 1.5 miles per hour, the vehicle would have to travel 1,050 feet.

If needed, similar curves can be prepared for trucks with other weight-power ratios or for other entering speeds from the results of motor-vehicle performance studies conducted by the Bureau of Public Roads and others (*3–6*). This was not necessary for the West Virginia needs study because the truck was operated over all routes under consideration.

Table 7.—Average speed of typical truck entering grades at a speed of 40 miles per hour

Gradient	Average speeds on grades extending—												Sustained speed on grade	Distance required to reach sustained speed
	0.1 mile	0.2 mile	0.4 mile	0.6 mile	0.8 mile	1.0 mile	1.5 miles	2.0 miles	3.0 miles	4.0 miles	5.0 miles	6.0 miles		
Pct.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. y.*	*M p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*Miles*
3	37.3	34.6	28.4	24.6	21.9	20.4	18.7	17.9	17.3	16.9	16.7	16.6	16.0	0.78
4	36.1	31.7	23.4	18.5	16.6	15.7	14.6	14.1	13.6	13.4	13.3	13.2	12.8	.60
5	35.2	29.3	18.2	14.9	13.7	13.1	12.3	11.9	11.6	11.5	11.4	11.3	11.0	.37
6	34.0	25.8	14.5	12.4	11.5	11.0	10.5	10.2	10.0	9.8	9.8	9.7	9.5	.28
7	32.6	21.4	11.8	10.2	9.5	9.2	8.8	8.5	8.4	8.3	8.2	8.2	8.0	.24

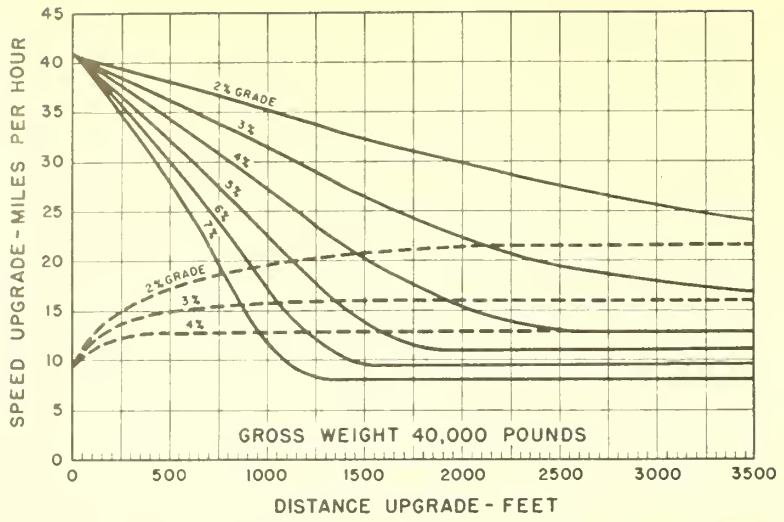

Figure 8.—Effect of length of grade on the speed of the control truck.

If the grades had been uniform and their length and gradient known, it would have been possible to determine the average truck factor by applying the data from figure 7 to figure 6. Driving the truck over the routes would have been unnecessary. This method was employed in Kentucky and Tennessee. In West Virginia, however, the needed information for the grades was not available. Furthermore, in this State there are few uniform grades. Practically all have multiple gradients for which it is possible, but rather difficult and time-consuming, to calculate truck speeds accurately. One such example is shown in figure 9. Also shown is the speed profile recorded for the control truck.

Truck Climbing Lanes Increase Capacity

Truck climbing lanes on the uphill side of long steep grades, as shown in figure 10, provide a means for improving the capacity of two-lane roads through rough or mountainous terrain. It is on a long steep grade that the greatest difference occurs between the normal speed of passenger cars and the normal speed of trucks. The need for adequate passing opportunities is therefore greatest on the long steep grades, whereas the passing opportunities are generally less than on the level sections of a two-lane highway. This results in higher truck factors and lower capacities for uphill sections of a two-lane highway than for the level sections.

Where truck climbing lanes are provided, the truck factor becomes zero and the capacity of the normal section of the two-lane highway is the same as though there were no trucks. Under certain conditions, therefore, truck climbing lanes will increase the practical capacity of an entire two-lane highway to a value higher than that for the same alinement with no grades. This is because the provision of a climbing lane reduces the average truck factor and increases the percentage of the highway on which passing maneuvers may be performed.

Climbing lanes will also increase the capacity of multilane highways. In fact, an added lane for each direction of travel over the entire length of a multilane highway may often be avoided by providing an added lane on the uphill side of long or steep grades.

The quantitative effect that trucks have on the capacity of multilane highways with long steep grades is not as well known, however, as for two-lane highways. For example, it is entirely possible that a few heavy trucks on a long steep grade of a multilane highway might have nearly as great an effect as a much larger number. The factors used at present are average values determined for less than 20 percent dual-tired vehicles—usually 5 to 10 percent.

Application of Uphill Truck Lanes

The benefit to traffic by providing an uphill truck lane at a specific location depends upon the following factors: (1) traffic volume, (2) percentage of trucks, (3) length and steepness of grade, and (4) availability of passing sight distance.

The information in table 8 offers some guidance for the application of climbing lanes. The fourth column in this table, for example, shows the lengths of grade for an average truck speed of 34 miles per hour or a truck factor of 3.0. At this average speed, even

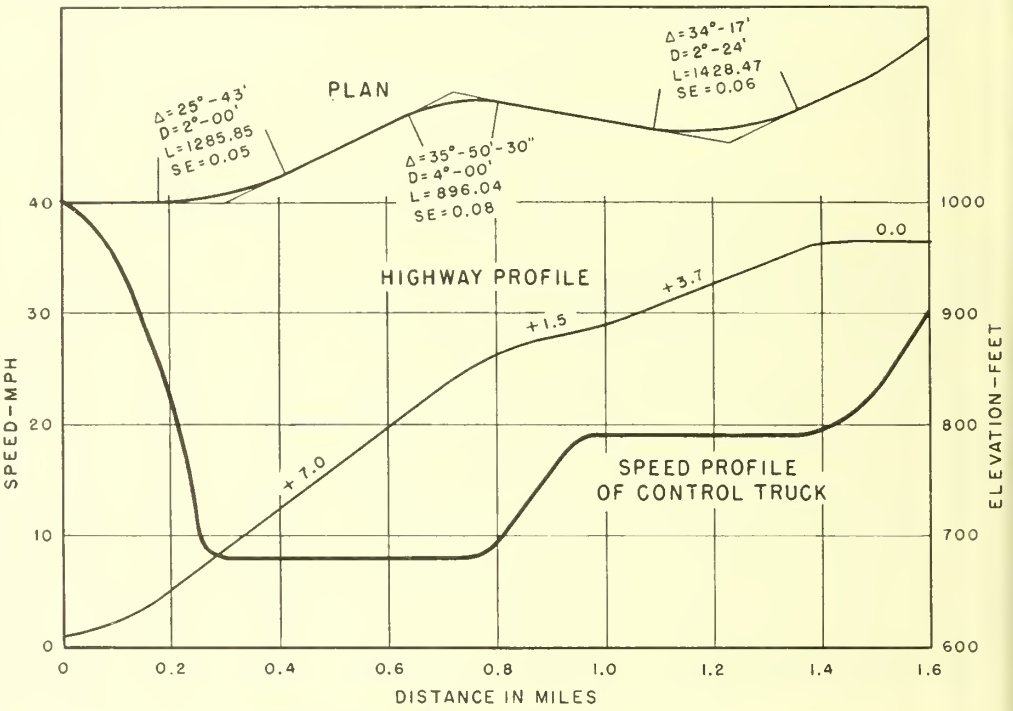

Figure 9.—Speed profile of the control truck.

Figure 10.—Truck climbing lane on U. S. Route 40, south of Middletown, Md.

Table 8.—Speed characteristics of control truck on upgrades, when entering grade from level section at 40 miles per hour

Gradient	Crawl speed [1]		Distance upgrade for an average speed of—		
	Velocity	Distance upgrade	34 m.p.h. or a truck factor of 3.0	27 m.p.h. or a truck factor of 6.5	19 m.p.h. or a truck factor of 13.8
Pct.	*M.p.h.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*
3	16.0	4,000	1,100	2,000	6,600
4	12.8	2,600	800	1,500	3,000
5	11.0	1,800	600	1,200	2,000
6	9.5	1,500	500	1,000	1,500
7	8.0	1,300	400	800	1,200

[1] Speed which truck can maintain indefinitely.

though about half of the trucks will be traveling at somewhat lower speeds, the speeds of passenger cars will not be affected sufficiently to greatly inconvenience the drivers. At traffic volumes approaching practical capacities for level sections of two-lane highways, few passenger cars will overtake a truck on grades that are shorter than those shown in column 4. For those that do, the necessary reduction in speed and the lost time in reaching the top of the grade when the passing sight distance is restricted will not be appreciably greater than commonly necessary due to oncoming traffic on straight level sections. Truck climbing lanes cannot be justified, therefore, on grades shorter than those shown in the fourth column of table 8.

The fifth and sixth columns of table 8 show lengths of grade on which there is the same relative need for a truck climbing lane. With a given traffic volume, for example, there is the same need for a climbing lane on a 3-percent grade 2,000 feet long as on a 7-percent grade 800 feet long.

The capacities of two-lane highways on grades with and without truck climbing lanes are shown in table 9 for the conditions applicable to West Virginia. The various groups shown for the length of grade in the second column are purely arbitrary with the exception of the shortest length shown for each gradient. The grades could have been divided into a larger or smaller number of length groups with corresponding changes in the average annual traffic volumes. The number of groups that have been used are believed to be consistent with the accuracy justified by the analyzed data.

Table 9 is based on the assumption that each climbing lane will be continuous from a point near the bottom of the grade to a point beyond the top of the grade where the sight distance becomes unrestricted and truck speeds again approach those of passenger cars. All steep grades of equal gradient longer than 4,000 feet have the same capacities. Prior to traveling 4,000 feet upgrade, most trucks will have reached their crawl speeds.

For certain traffic and terrain conditions on exceedingly long grades, the use of passing bays may be an adequate and a more feasible solution than a continuous climbing lane (*5*). With passing bays, the capacity of a two-lane road would be greater than without the passing bays and, for certain conditions, might equal the capacities shown in table 9 for the two-lane roads with a truck lane. The maximum capacities with continuous truck lanes are actually higher than most of the values in table 9. It was assumed that the capacity on a grade with a truck lane could not exceed the capacity of a two-lane level section. The capacity with a truck lane falls below the capacity of a level section only on the long grades over 5 percent where downhill speeds of trucks traveling in the lower gears affect capacities.

Application to Capacity Determinations

The tables and charts that have been presented are the basic information needed for capacity determinations in connection with the West Virginia needs studies. From this information, an almost unlimited number of special tables and charts can be prepared for specific conditions in either West Virginia or other States. The data can also be applied in many different ways as will be explained by the applications made for the West Virginia, Kentucky, and Tennessee studies.

In order to determine the highway needs in West Virginia, it was necessary to have a vast amount of information concerning the roads and the traffic using them. For the capacity determination with which this report is concerned, only the factors that have been previously discussed were needed. Their effect on the capacities of two-lane roads can be determined from tables 1, 5, and 9, and figure 6. Figure 7 was also needed for the Kentucky and Tennessee studies since a control truck was not used to determine the truck factors in these States.

It is important that the conditions be similar over a length of highway for which a capacity determination is made. Section limits, for this reason, were usually defined by urban limits; or by a change in the traffic volume, surface width, average highway speed, or type of terrain; or by a marked change in the percentage of highway with a 1,500-foot passing sight distance. In addition, a county line was the end of one section and the beginning of another.

Application to West Virginia highways

Five typical sections analyzed during the West Virginia studies will illustrate the procedures used to apply the capacity information. The basic information and the resulting calculations for each of these sections are shown in table 10.

Section 1 as shown in table 10 is located on U. S. Route 60 about 20 miles west of Charleston in Putnam County. It is 6.4 miles long with a 26-foot pavement in rolling terrain. It has an excellent passing distance as compared with most West Virginia roads, since 59 percent of its length has a sight distance in excess of 1,500 feet. The average highway speed is 65 miles per hour, and the generally flat profile results in a truck equivalent of only 2. The average daily traffic volume was 5,500 in 1955 with a design-hour factor of 12 percent of the ADT having 7 percent trucks.

The capacity of this section is 5,800 vehicles daily at an operating speed of 45–50 miles per hour, or 7,150 vehicles daily at a tolerable operating speed of 40–45 miles per hour. As U. S. Route 60 is one of the most important highways in the State, it is desirable to provide conditions conducive to a high operating speed.

For an operating speed of 45–50 miles per hour the existing traffic volume is practically equal to the capacity of the section. As it would be impractical to attempt to increase the capacity of the existing road by improving passing sight distances, the only recourse to accommodate expected future traffic volumes is to add additional lanes by constructing

Table 9.—Capacities of two-lane highways on grades carrying 5-percent truck traffic, based on a 30th highest hour of 12 percent of the average annual traffic volume

Gradient	Length of grade	Average annual traffic volumes							
		Average highway speed, 70 m. p. h.; operating speed, 50–55 m. p. h.		Average highway speed, 60–70 m. p. h.; operating speed, 50 m. p. h.		Average highway speed, 60–70 m. p. h.; operating speed, 45–50 m. p. h.		Average highway speed, 50–70 m. p. h.; operating speed, 40–55 m. p. h.	
		Without truck lane	With truck lane	Without truck lane	With truck lane	Without truck lane	With truck lane	Without truck lane	With truck lane
Pct	*Ft.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*
3	1,100–2,000	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	2,000–4,000	3,850	4,700	4,300	5,500	4,550	6,000	6,250	7,000
	Over 4,000	3,500	4,700	3,800	5,500	4,050	6,000	5,550	7,000
4	800–1,500	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,500–3,000	3,850	4,700	4,200	5,500	4,400	6,000	5,800	7,000
	3,000–4,000	3,400	4,700	3,750	5,500	4,000	6,000	5,450	7,000
	Over 4,000	3,200	4,700	3,400	5,500	3,800	6,000	5,100	7,000
5	600–1,200	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,200–2,000	3,500	4,700	3,800	5,500	4,050	6,000	6,150	7,000
	2,000–4,000	3,200	4,700	3,700	5,500	3,950	6,000	5,700	7,000
	Over 4,000	2,800	4,700	3,250	5,500	3,500	6,000	4,800	7,000
6	500–1,000	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,000–1,500	3,500	4,700	3,800	5,500	4,050	6,000	6,150	7,000
	1,500–4,000	3,050	4,200	3,550	5,200	3,800	6,000	5,350	7,000
	Over 4,000	2,550	3,600	2,950	4,000	3,200	4,200	4,500	5,800
7	400–800	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	800–1,200	3,500	4,700	3,800	5,500	4,050	6,000	5,550	7,000
	1,200–2,500	2,900	4,200	3,400	5,000	3,650	6,000	5,100	7,000
	2,500–4,000	2,600	3,600	3,000	4,000	3,300	4,200	4,600	5,800
	Over 4,000	2,000	3,000	2,400	3,400	2,650	3,600	3,700	4,600

Table 10.—Capacity analysis of typical highway sections in West Virginia

	Section No. 1	Section No. 2	Section No. 3	Section No. 4	Section No. 5
KNOWN CONDITIONS					
Route designation	U. S. 60	U. S. 21	U. S. 60	U. S. 50	U. S. 50
Length of section—miles	6.37	6.21	7.00	4.13	11.46
Type of terrain	Rolling	Rolling	Rolling	Rolling	Rolling
Average highway speed—m. p. h.	65	40	53	57	53
1,500-foot sight distance, percent available	59	2	10	9	15
Surface width—ft.	26	18	22	22	20
30th-highest-hour factor—pct.	12	12	12	13	14
Truck speed—m. p. h.	40	32	29	25	24
Commercial vehicles—pct.	7	5	5	5	5
1955 average daily traffic	5,500	2,300	2,400	2,400	2,600
Long grades	None	None	(1)	(1)	(1)
Sharp curves	None	None	(2)	None	(2)
DETERMINED VALUES					
Tolerable operating speed [3]—m. p. h.	40–45	40–45	40–45	40–45	40–45
Tolerable design speed [3]—m. p. h.	50	50	50	50	50
Tolerable capacity [4]—ADT	7,300	None	3,070	3,800	3,405
Width factor	1.00	.70	.86	.86	.77
30th-hour factor	1.00	1.00	1.00	.92	.86
Truck equivalent	2.0	4.0	5.0	8.0	9.0
Truck factor	.98	.91	.88	.78	.75
Corrected tolerable capacity [5]—ADT	7,150	None	2,300	2,350	1,700

[1] Section 3, 0.40 mile at 8 percent; section 4, 0.50 mile at 7.5 percent, 0.35 mile at 4 percent, 0.60 mile at 2.5 percent; section 5, 0.5 mile at 6.5 percent, 1.2 miles at 5 percent, 2.5 miles at 2.5 percent.
[2] Section 3, three curves at 30, 35, and 45 miles per hour, and two curves at 50 miles per hour; section 5, one curve at 40, one at 45, and three at 50 miles per hour.
[3] Taken from table 1.
[4] Taken from table 5.
[5] Computed as follows: 105+(100−percentage of commercial vehicles)+(percentage of commercial vehicles × truck equivalent).

another one-way roadway and using the existing lanes for the other direction of travel.

Section 2 on U. S. Route 21 in Jackson County is also on one of the more important roads in the State, although the traffic volume is not high. The 6.2-mile section is located in rolling terrain about 20 miles north of Charleston. Both the alinement and profile are poor, resulting in a low design speed and limited sight distance. Pavement width is 18 feet and the truck equivalent is 4. The traffic volume during 1955 was 2,300 vehicles per day with the design-hour factor of 12 percent of the ADT with 5 percent trucks.

The road as it exists today does not meet the tolerable standards for this class of highway. Since its average highway speed is only 40 miles per hour, it cannot carry traffic at the tolerable speed of 50 miles per hour during low volumes nor at 40 to 45 miles per hour during the 30th highest hourly volume of the year. It therefore has no capacity for these speeds.

Some improvement in alinement could be made to increase the average highway speed and the amount of passing sight distance. By providing a 1,500-foot sight distance over 10 percent of the length and raising the average highway speed to 45 miles per hour, the capacity would be increased to 2,200 vehicles per day at an operating speed of 35–40 miles per hour, or to 1,050 vehicles per day at a 40- to 45-miles-per-hour operating speed.

Widening the entire section to 24 feet would increase the capacities at the 35- to 40- and 40- to 45-miles-per-hour operating speeds to 3,100 and 1,600 vehicles per day, respectively.

Since the tolerable operating speed for this highway is 40–45 miles per hour, the 35- to 40-miles-per-hour operating speed would be inadequate and undesirable. The capacity at a minimum desirable operating speed with the alinement and sight distances improved to the extent possible on the existing location is still considerably less than the existing traffic volume. The conclusion is, therefore, that the only lasting solution is a complete redesign of the highway.

Section 3 is located on U. S. Route 60 in Greenbrier County about 100 miles east of Charleston. The terrain is rolling over this 7.0-mile section, the average highway speed is 53 miles per hour, about 10 percent of the highway has a 1,500-foot sight distance, and the truck equivalent is 5. Tolerable operating speed for this highway is 40–45 miles per hour. At this speed the capacity of the section is 2,300 vehicles per day.

Several possibilities are available for increasing the capacity of the section, including removal of some or all of the five substandard curves, the addition of truck lanes on grades, and minor improvements in the sight distance by removal of trees, daylighting curves, etc.

Reducing curvatures would increase the average highway speed to about 55 miles per hour, resulting in a tolerable capacity of about 2,600 vehicles per day. Passing sight distance might be increased an additional 5 percent by miscellaneous measures, such as brush removal, curve daylighting, etc. This would further increase capacity to about 2,800 vehicles per day.

The next alternative is the provision of truck lanes. The existing grades would require about one mile of truck lanes to be added, resulting in a decrease in the overall truck equivalent from 5 to 3. Minor improvement in the alinement would also provide additional passing sight distance so that a 1,500-foot sight distance would be available over approximately 20 percent of the highway. All these improvements would increase the capacity of the highway to about 3,380 vehicles per day at an operating speed of 40–45 miles per hour. At the normal rate of traffic growth, this volume would not be exceeded for a period of 6 or 7 years. Thereafter it would be necessary to undertake major changes in the alinement or to provide a four-lane highway in order to maintain the desired operating speed.

Section 4 has 11-foot traffic lanes and is located on U. S. Route 50 in Wood County. It is a 4.1-mile section through rolling terrain. The alinement is fairly good since the average highway speed is 57 miles per hour and 9 percent of the highway has a 1,500-foot sight distance. The traffic volume in 1955 was 2,400 vehicles per day with a design-hour factor of 13 percent of the ADT including 5 percent trucks.

For the tolerable operating speed of 40–45 miles per hour, the capacity is 2,350 vehicles per day. This is slightly lower than the present volume. Building a truck lane one mile long on a critical grade would reduce the truck equivalent to 3 and would increase the 1,500-foot passing sight distance from 9 percent to about 18 percent of the length. As a result the capacity would be increased to 3,250 vehicles per day at an operating speed of 40–45 miles per hour. Some additional 1,500-foot sight distance could be obtained by increasing the view on the inside of several curves by simply removing the obstructions such as brush and low banks on the right-of-way. When the obstruction is off the right-of-way, additional right-of-way must be purchased or an agreement reached with the property owner to keep it cleared. An additional 5 percent of 1,500-foot sight distance can be obtained in this manner. This would increase the capacity at the desired operating speed to 3,450 vehicles per day, which represents an increase of nearly 60 percent over the present traffic volume, or to approximately the volume expected in 1970.

Section 5 is located on U. S. Route 50 in Hampshire County in the northeastern part of the State. The section is 11.5 miles long with uniform design characteristics in the rolling terrain. The average highway speed is 53 miles per hour, the surface width is 20 feet, and 15 percent of the highway has a 1,500-foot sight distance. The truck equivalent is 9. The present ADT is 2,600 per day with a design-hour factor of 14 percent including 5 percent trucks. Under these conditions, the capacity at an operating speed of 40–45 miles per hour is 1,700 vehicles per day.

Several possibilities exist for improving the capacity. These include reducing the sharpness of five substandard curves, widening the surface, the addition of truck lanes on grades, and minor improvement in the sight distance. Widening from 20 to 24 feet would increase the capacity to 2,200 vehicles per day. Removal of the substandard curves will increase the average highway speed to about 55 miles per hour and would increase the passing sight distance 1 to 2 percent. These improvements, including the widening, would result in increasing the capacity to 2,550 vehicles per day.

The addition of 2½ miles of truck lanes would increase the sections on which passings could be performed to about 25 percent of the highway and reduce the truck equivalent to 3. The total resulting capacity would be 3,600 vehicles per day or 38 percent above the present volume.

These five examples are rather typical of the way the capacity information was applied in West Virginia to determine highway sufficiency. Its use was found especially helpful in pointing out the changes that could be made to improve capacity. Altering some highway features will have little effect on the capacity at a desired operating speed, while others, such as the provision of truck lanes and substantially improving the passing sight distances, will have a major effect.

Application to Kentucky and Tennessee highways

The principles employed for capacity determinations in West Virginia have general application wherever curvatures and grades create special highway capacity problems. This was the case throughout most of Kentucky and Tennessee where highway needs studies were started during the period that the West Virginia study was being completed.

The two special features needed in the refinement of the capacity analysis, which were the average highway speed and the truck equivalent, could have been obtained in the same manner as described for West Virginia. Utilizing the experience gained in the West Virginia study, however, it was found desirable and more feasible to derive these data from existing records rather than from test vehicle operation.

Kentucky and Tennessee lacked data on actual truck operations which would be consistent with probable future conditions. Following many years of severe restrictions on truck sizes and weights, Tennessee had just revised its law so as to be in substantial agreement with AASHO recommendations. Truck operations, however, had not as yet changed to conform with the higher limits. Kentucky still retained its low limits, but it was anticipated that a more realistic position would be adopted—as it was in 1956—bringing that State in line with Tennessee and the other States. Without actual data on vehicle weights for the revised weight limits, it was assumed that future conditions in Kentucky and Tennessee would be similar to those on which the West Virginia study was based.

In both Kentucky and Tennessee, geometric design data were available, mile by mile, in the State Highway Planning Division records, or were easily obtainable from plans. Thus actual curvature was known, and curve lengths could be obtained or sampled from the plans. In both States, the gradient and the length of the grades on each section of highway were available from the plans. This was not the case for most roads in West Virginia.

Alternate method used for determining average highway speed in Kentucky and Tennessee.—Operation of a test car, as in West Virginia, accounted for several factors that would affect the average highway speed, but horizontal curvature was by far the most significant. From available data, therefore, it was possible to approximate the average highway speed of control sections in the other States by concentrating the analysis on the combined effect of horizontal curves and tangents.

Vehicle speeds are affected ahead and beyond a curve for a distance which varies with the degree of curvature. That is, a vehicle on a tangent approaching a sharp curve must begin to slow down before reaching the curve in order to reduce its speed to the allowable speed on the curve. After traveling around the curve, additional time and distance are required to accelerate back to the normal tangent speed. It was therefore necessary to determine the following information for each section of highway requiring a separate capacity analysis:

1. The possible safe speed, or design speed, of each curve.
2. The length of each curve.
3. The distance before and after each curve that the speed was affected, together with the average speed while decelerating and accelerating.
4. The average speed weighted by the length of the tangents, the curves, and by the deceleration and acceleration distances. This speed was used as the average highway speed.

The safe speeds for curves of various degrees (or radii) were determined from the tables in the AASHO policy on *Geometric Design of Rural Highways* (*1*). The length of each curve was obtained from the highway plans or from planning survey information. Comfortable rates of acceleration and deceleration as shown in the AASHO policies were used to determine the length of speed transitions between the curves and tangents.

A special study conducted by sampling the curves on level sections from Kentucky highway plans showed that, regardless of curvature, the average total effect of a curve on the speed of a vehicle was equivalent to a travel distance of about 800 feet at the safe speed for the curve. For example, the 9-degree curves good for a design speed of 45 miles per hour had an average length of 667 feet. Decelerating and accelerating from the 65 miles per hour tangent speed required a total of 485 feet. On an average, a vehicle would be affected for a total distance of 1,152 feet but the time lost was the same as if the speed was 45 miles per hour for 915 feet and the tangent speed was 65 miles per hour on the rest of the section. Likewise for the 40-degree curves, the equivalent distance at 20 miles per hour was 691 feet. The equivalent distances varied from curve to curve but the average was 780 feet with values much greater or less than the average being comparatively rare. An equivalent length of 800 feet or 0.15 mile for all curves was therefore used to determine the average highway speeds for the highway sections in Kentucky and Tennessee.

Tangent sections and curves as sharp as 3 degrees were assumed to have a highway speed of 70 miles per hour if there were no curves as sharp as 4 degrees on the highway. If any curves on the highway were as sharp as 4 degrees, the tangent sections and the curves of 4 degrees or flatter were assumed to have a highway speed of 65 miles per hour. These assumptions are in accordance with the AASHO definition of design speed as related to the travel speeds found on main rural highways during low traffic densities.

Table 11.—Illustration of method used in estimating the average highway speed of a two-lane highway

Curvature	Safe speed	Number of curves	Total length	Product of columns 2 and 4
Degrees	*M. p. h.*		*Miles*	
6	55	1	0.15	8.25
10	43	2	.30	12.90
12	40	8	1.20	48.00
20	35	4	.60	21.00
30	25	1	.15	3.75
0	65	0	7.60	494.00
Total	------	-------	10.00	587.90
Average highway speed-----587.90÷10.00=59 m. p. h.				

Table 11 illustrates the method used in estimating the average highway speed of a two-lane section of highway 10 miles long. If weighted by travel time the average highway speed would be 56 miles per hour. Within the limits of reasonable accuracy, however, either method should be satisfactory. For the needs studies conducted in Ontario, Canada, the weighting to obtain the average highway speed was done on the basis of time involved rather than length.

Method used for determining truck equivalent in Kentucky and Tennessee.—The first section of this article calls attention to the fact that driving a test truck to establish a speed profile would be unnecessary if gradient and length were known, since available test data are adequate to establish truck speeds on known grades (fig. 8).

With grade data available in Kentucky and Tennessee, the truck equivalent in terms of passenger cars, for capacity computations, was determined from figures 7 and 6, in that order.

It was first assumed that the entering speed of trucks approaching a grade was 40 miles per hour. It is recognized that momentum from downgrades and actual level speeds may frequently be greater, but in the mountainous terrain where this analysis was especially pertinent, horizontal curvature is such that higher speeds are seldom encountered. For example, the speed profile of the test truck on U. S. Route 50 in West Virginia shows a maximum of only 45 miles per hour for short distances at only three locations in a 50-mile section.

It was also assumed for the purposes of this study that the average truck speed was 40 miles per hour on level terrain, on all grades of less than 3 percent, and on grades of 3 percent less than 500 feet long. For all other grades, the average truck speed was determined from curves in figure 7 for each grade or average compound grade in one direction only.

For the control section or a long subsection, the average truck speed was determined by weighting by distance the speeds on level terrain and the several grades. Finally, the

(Continued on page 44)

Table A.—Tolerable capacities of existing two-lane highways located in flat terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	7,100	7,100	6,600	5,650	6,100	6,100	5,700	4,850	5,450	5,450	5,100	4,350	4,950	4,950	4,600	3,950
80	90	6,800	6,400	5,750	5,050	5,850	5,500	4,950	4,350	5,250	4,950	4,450	3,900	4,750	4,500	4,000	3,550
60	80	6,400	5,550	4,800	4,000	5,500	4,750	4,150	3,450	4,950	4,250	3,700	3,100	4,500	3,900	3,350	2,800
40	70	5,750	4,600	3,900	2,800	4,950	3,950	3,350	2,400	4,450	3,550	3,000	2,150	4,000	3,200	2,750	1,950
20	60	4,900	3,750	2,800	2,000	4,200	3,200	2,400	1,700	3,750	2,900	2,150	1,550	3,450	2,600	1,950	1,400
0	50	3,800	2,800	2,000	1,250	3,250	2,400	1,750	1,050	2,900	2,150	1,550	950	2,650	1,950	1,400	850

Table B.—Tolerable capacities of existing two-lane highways located in flat terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	8,450	8,050	7,450	6,550	7,250	6,900	6,400	5,650	6,500	6,200	5,750	5,050	5,900	5,650	5,200	4,600
80	87	7,700	7,300	6,700	5,800	6,600	6,300	5,750	5,000	5,950	5,600	5,150	4,450	5,400	5,100	4,700	4,050
60	76	6,800	6,450	5,700	4,850	5,850	5,550	4,900	4,150	5,250	4,950	4,400	3,750	4,750	4,500	4,000	3,400
40	64	5,900	5,300	4,600	3,700	5,050	4,550	3,950	3,200	4,550	4,100	3,550	2,850	4,150	3,700	3,200	2,600
20	52	4,950	4,100	3,200	2,200	4,250	3,550	2,750	1,900	3,800	3,150	2,450	1,700	3,450	2,850	2,250	1,550
0	40	3,950	2,700	1,950	1,250	3,400	2,300	1,700	1,100	3,050	2,100	1,500	950	2,750	1,900	1,350	900

Table C.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	6,500	6,500	6,050	5,150	5,600	5,600	5,200	4,450	5,000	5,000	4,650	3,950	4,550	4,550	4,250	3,600
80	87	6,200	5,850	5,250	4,600	5,350	5,050	4,500	3,950	4,750	4,500	4,050	3,550	4,350	4,100	3,700	3,200
60	76	5,850	5,050	4,400	3,650	5,050	4,350	3,800	3,150	4,500	3,900	3,400	2,800	4,100	3,550	3,100	2,550
40	64	5,250	4,200	3,550	2,550	4,500	3,600	3,050	2,200	4,050	3,250	2,750	1,950	3,700	2,950	2,500	1,800
20	52	4,500	3,460	2,550	1,800	3,850	2,950	2,200	1,550	3,450	2,600	1,950	1,400	3,150	2,400	1,800	1,250
0	40	3,450	2,550	1,850	1,150	3,000	2,200	1,600	1,000	2,650	1,950	1,400	900	2,400	1,800	1,300	800

Table D.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	7,700	7,350	6,800	6,000	6,600	6,300	5,850	5,150	5,950	5,650	5,250	4,600	5,400	5,150	4,750	4,200
80	85	7,050	6,650	6,100	5,300	6,050	5,700	5,250	4,550	5,450	5,100	4,700	4,100	4,950	4,650	4,250	3,700
60	72	6,200	5,900	5,200	4,400	5,350	5,050	4,450	3,800	4,750	4,550	4,000	3,400	4,350	4,150	3,650	3,100
40	58	5,400	4,850	4,200	3,400	4,650	4,150	3,600	2,900	4,150	3,750	3,250	2,600	3,800	3,400	2,950	2,400
20	44	4,500	3,750	2,900	2,000	3,850	3,200	2,500	1,700	3,450	2,900	2,250	1,550	3,150	2,600	2,050	1,400
0	30	3,600	2,450	1,800	1,150	3,100	2,100	1,550	1,000	2,750	1,900	1,400	900	2,500	1,700	1,250	800

Table E.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 35–40 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.
100	100	9,050	8,750	8,300	7,200	7,800	7,500	7,150	6,200	7,000	6,750	6,400	5,550	6,350	6,100	5,800	5,050
80	83	8,200	8,000	7,500	6,500	7,050	6,900	6,450	5,600	6,300	6,150	5,800	5,000	5,750	5,600	5,250	4,550
60	67	7,400	7,250	6,600	5,500	6,350	6,250	5,700	4,750	5,700	5,600	5,100	4,250	5,200	5,100	4,620	3,850
40	51	6,500	6,350	5,600	4,300	5,600	5,450	4,800	3,700	5,000	4,900	4,300	3,300	4,550	4,450	3,920	3,000
20	36	5,750	5,350	4,100	2,850	4,950	4,600	3,500	2,450	4,400	4,100	3,150	2,200	4,000	3,750	2,900	2,000
0	20	4,900	3,500	2,250	1,400	4,200	3,000	1,950	1,200	3,800	2,700	1,750	1,100	3,450	2,450	1,600	1,000

Table F.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	5,500	5,500	5,150	4,400	4,750	4,750	4,450	3,800	4,250	4,250	3,950	3,400	3,850	3,850	3,600	3,100
80	87	5,300	5,000	4,450	3,900	4,550	4,300	3,850	3,350	4,100	3,850	3,450	3,000	3,700	3,500	3,100	2,750
60	76	5,000	4,300	3,750	3,100	4,300	3,700	3,200	2,650	3,850	3,300	2,900	2,400	3,500	3,000	2,600	2,150
40	64	4,450	3,550	3,050	2,200	3,850	3,050	2,600	1,900	3,450	2,750	2,350	1,700	3,100	2,500	2,150	1,550
20	52	3,800	2,900	2,200	1,550	3,250	2,500	1,900	1,350	2,950	2,250	1,700	1,200	2,650	2,050	1,550	1,100
0	40	2,950	2,200	1,550	950	2,550	1,900	1,350	800	2,250	1,700	1,200	750	2,050	1,550	1,100	650

Table G.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	6,550	6,250	5,800	5,100	5,650	5,350	5,000	4,400	5,050	4,800	4,450	3,950	4,600	4,350	4,050	3,550
80	85	6,000	5,700	5,200	4,500	5,150	4,900	4,450	3,850	4,600	4,400	4,000	3,450	4,200	4,000	3,650	3,150
60	72	5,300	5,000	4,450	3,750	4,550	4,300	3,850	3,200	4,100	3,850	3,450	2,900	3,700	3,500	3,100	2,600
40	58	4,600	4,100	3,600	2,900	3,950	3,550	3,100	2,500	3,550	3,150	2,750	2,250	3,200	2,850	2,500	2,050
20	44	3,850	3,200	2,500	1,700	3,300	2,750	2,150	1,450	2,950	2,450	1,900	1,300	2,700	2,250	1,750	1,200
0	30	3,050	2,100	1,500	950	2,600	1,800	1,300	800	2,350	1,600	1,150	750	2,150	1,450	1,050	650

Table H.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 35–40 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.
100	100	7,700	7,450	7,050	6,150	6,600	6,400	6,050	5,300	5,950	5,750	5,450	4,750	5,400	5,200	4,950	4,300
80	83	7,000	6,800	6,350	5,550	6,000	5,850	5,450	4,750	5,400	5,250	4,900	4,250	4,900	4,750	4,450	3,900
60	67	6,300	6,200	5,650	4,650	5,400	5,350	4,850	4,000	4,850	4,750	4,350	3,600	4,400	4,350	3,950	3,250
40	51	5,550	5,400	4,800	3,650	4,750	4,650	4,150	3,150	4,250	4,150	3,700	2,800	3,900	3,800	3,350	2,550
20	36	4,900	4,550	3,500	2,400	4,200	3,960	3,000	2,050	3,750	3,500	2,700	1,850	3,450	3,200	2,450	1,700
0	20	4,150	3,000	1,900	1,150	3,550	2,600	1,650	1,000	3,200	2,300	1,450	900	2,900	2,100	1,350	800

The Economic Costs of Motor-Vehicle Accidents of Different Types

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by ROBIE DUNMAN, Transportation Economist

This article discusses the frequencies and direct costs of different types of accidents involving passenger cars in Massachusetts during 1953.

Of the 131,500 accidents recorded in this study, nearly three-fourths were property-damage-only accidents, one-fourth were nonfatal-injury accidents, and less than half of one percent were fatal-injury accidents. For every dollar spent as a result of these accidents, nonfatal-injury accidents accounted for 57 cents; property-damage-only accidents, 40 cents; and fatal-injury accidents, 3 cents.

Collisions between passenger cars or passenger cars and other motor vehicles—by far the most frequent type of accident—accounted for 83 percent of the number and the same proportion of the cost of all accidents. Collisions with pedestrians, fixed objects, other objects, and noncollision types of accidents made up the remaining 17 percent.

Angle, rear-end, and head-on collisions represented nearly 81 percent of the number and 89 percent of the cost of all collisions between passenger cars or passenger cars and other motor vehicles. Angle collisions were the most frequent and were followed in order by rear-end and head-on collisions.

IN December 1947 the Highway Research Board recommended that the Bureau of Public Roads cooperate with the States in conducting studies of the economic costs of motor-vehicle accidents. These studies are now underway in Massachusetts, New Mexico, and Utah, and a fourth study is programed in Wisconsin. On the basis of preliminary discussion, it is anticipated that a fifth study will be started in Michigan during 1958.

This article reports data developed by the Massachusetts Department of Public Works and by the Massachusetts Registry of Motor Vehicles in cooperation with the Bureau of Public Roads. It is emphasized that the findings may not be typical or average for all States.

The number and direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953 are shown by type of accident and severity of accident. Comparisons are made on the basis of cost per accident, per capita, per passenger car registered, per licensed operator, per mile of road, and per 100 million vehicle-miles of travel.

The purpose of this article is to present the cost of accidents in relation to the types of accidents in a way that will be helpful to groups and individuals who are trying to reduce traffic accidents and the resulting economic loss.

Statistical studies of the economic cost of motor-vehicle accidents are based on a probability sample of the accident experience of vehicle owners. They are designed to be accurate within 10 percent. By means of mailed questionnaires and through personal interviews with selected vehicle owners, their accident experience for one year is obtained. From these data the direct cost of accidents is estimated and correlated with the more important characteristics of accidents, including those peculiar to the highway and street facilities, the driver, and the vehicle. These studies are statewide and comprehensive. Because the data collected and analyzed in each State are so detailed and voluminous, this article is confined to one segment of the comprehensive study of traffic-accident costs in Massachusetts during 1953 and relates only to accidents in which passenger cars were involved. The accidents were motor-vehicle traffic accidents occurring on public roadways and involving motion.

Definitions

Direct costs are defined as the money value of damages and losses to persons and property that were the direct result of these accidents and which might have been saved had these accidents not occurred. Direct costs are composed of the money value of damage to property; hospitalization; services of physicians, dentists, and nurses; ambulance use; medicine; work time lost; damages awarded in excess of other direct costs; attorneys' services; court fees; and other miscellaneous but small items.

The type of collision was determined by the direction of travel of the vehicles involved before the collision, and not by what took place because of efforts on the part of drivers to avoid collision. Thus, any collision involving an intentional change of direction such as a right, left, or U-turn was classified as a turning movement, even though this may have resulted in a head-on or rear-end type of collision. Similarly, an angle collision resulted when two or more vehicles, each traveling in a straight line, came together at an intersection. Although one or both drivers may have swerved to avoid impact and collided in a sideswipe fashion, this was still classified as an angle collision. On the same basis, a collision involving a vehicle entering or leaving a parking space was classified as a parking maneuver, even though the vehicle struck, or was struck by another, in a head-on, rear-end, or sideswipe fashion. A sideswipe occurred only when vehicles collided while overtaking and passing in the same direction, passing in the opposite direction, or passing a parked vehicle.

All other definitions are from the manual *Uniform Definitions of Motor-Vehicle Accidents, 1947*, prepared under the auspices of the National Conference on Uniform Traffic Accident Statistics and published by the Federal Security Agency, U. S. Public Health Service.

In 1953 the population of Massachusetts was 4,773,000. There were 1,239,000 registered passenger cars and 1,858,000 licensed operators who drove their cars 11,628 million vehicle-miles over the Commonwealth's 24,500 miles of streets and highways. These passenger-car operators experienced 222,000 involvements in 131,500 accidents that resulted in a direct cost of $50,224,000.

Accident Severity

All motor-vehicle traffic accidents fall into one of three severity classes—property-damage-only accidents, nonfatal-injury accidents, and fatal-injury accidents.

Table 1 shows the accident experience of Massachusetts passenger-car operators during 1953 and brings into focus the numerical relation of accidents of different severity. The 33,270 nonfatal-injury accidents and the 315 fatal-injury accidents are in the ratio of 106: 1. This ratio is slightly more than 3 times the 35: 1 injury-to-fatal-accident ratio ordinarily used in estimating the cost of accidents. Whether or not this high ratio will hold in predominantly rural States will be known within a short time when results of the Utah and New Mexico studies are available.

It is apparent from table 1 that 3 out of 4 motor-vehicle traffic accidents resulted in property damage only, 1 out of 4 accidents resulted in a nonfatal injury, and only 1 in 417 accidents resulted in a fatal injury. It is also evident that there were 106 times as many nonfatal-injury accidents and 311 times as many property-damage-only accidents as there were fatal-injury accidents per 100 million vehicle-miles of travel.

On a population basis, there was 1 fatal

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

Table 1.—Motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by severity of accident

Number and rate of accidents	Severity of accident			All accidents
	Fatal injury	Nonfatal injury	Property damage only	
Number of accidents	315	33,270	97,915	131,500
Percent of total	0.2	25.3	74.5	100.0
Number of accidents per 100 million vehicle-miles of travel	[1] 3	286	842	1,131
Number of persons per accident	15,152	143	49	36
Number of registered passenger cars per accident	3,933	37	13	9
Number of licensed operators per accident	5,898	56	19	14

[1] Rounded from 2.7.

Table 2.—Direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by severity of accident

Severity of accident	Total direct cost	Percent of total	Total direct cost—					
			Per accident	Per capita	Per passenger car registered	Per licensed operator	Per mile of road	Per 100 million vehicle-miles of travel
	1,000 dollars							*1,000 dollars*
Fatal injury	1,642	3.3	$5,213	$0.34	$1.32	$0.88	$67	14
Nonfatal injury	28,688	57.1	862	6.01	23.15	15.44	1,171	247
Property damage only	19,894	39.6	203	4.17	16.06	10.71	812	171
All accidents	50,224	100.0	382	10.52	40.53	27.03	2,050	432

accident for every 15,152 persons, 1 nonfatal-injury accident for every 143 persons, and 1 property-damage-only accident for every 49 persons. The accident severity rate for passenger cars registered was as follows: 1 fatal accident for every 3,933 passenger cars, 1 nonfatal-injury accident for every 37 passenger cars, and 1 property-damage-only accident for every 13 passenger cars. The accident severity rate for licensed passenger-car drivers was 1 fatal-injury accident for every 5,898 drivers, 1 nonfatal-injury accident for every 56 drivers, and 1 property-damage-only accident for every 19 drivers.

The direct costs of motor-vehicle traffic accidents in Massachusetts are presented in table 2. It should be remembered in considering these costs that they apply only to accidents involving passenger cars, and they do not include any indirect costs such as the present value of future earnings and the overhead cost of motor-vehicle accident insurance.

After comparing the number of accidents with the costs shown in table 2, it is found that 1 fatal accident with a direct cost of $5,213 is the equivalent of either 6 nonfatal-injury accidents with a direct cost of $862 each or 25 property-damage-only accidents with a direct cost of $203 each. However, it is also apparent that on the basis of 100 million vehicle-miles of travel, property-damage-only accidents cost 12 times as much and nonfatal-injury accidents, 18 times as much as the cost of fatal accidents.

Furthermore, table 2 shows that from the economic point of view, nonfatal-injury accidents were by far the most significant. They accounted for 57 cents of every accident direct-cost dollar. Property-damage-only accidents accounted for 40 cents of every accident direct-cost dollar, whereas the emotion-packed and often highly dramatized fatal accident accounted for only 3 cents. This comparison does not minimize the personal tragedy of fatal accidents.

The relation of direct cost of accidents to population, licensed operators, passenger cars registered, and road mileages are also given in table 2.

The per capita cost ranged from 34 cents for fatal-injury accidents to $6.01 for nonfatal-injury accidents, and the total direct cost for all accidents averaged $10.52 per capita.

On the basis of direct cost per passenger-car registered, the range was from $1.32 for fatal-injury accidents to $23.15 for nonfatal-injury accidents. The total direct cost of all

Table 3.—Motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by type of accident

Number and rate of accidents	Passenger cars colliding with—				Non-collision accidents	All accidents
	Other motor vehicles	Pedestrians	Fixed objects	Other objects		
Number of accidents	109,700	5,900	7,300	6,400	2,200	131,500
Percent of total	83.4	4.5	5.5	4.9	1.7	100.0
Number of accidents per 100 million vehicle-miles of travel	943	51	63	55	19	1,131
Number of persons per accident	44	809	654	746	2,170	36
Number of passenger cars registered per accident	11	210	170	194	563	9
Number of licensed operators per accident	17	315	255	290	845	14

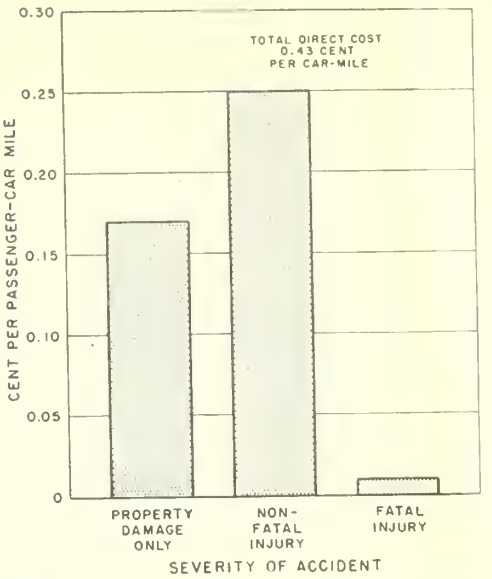

Figure 1.—Direct cost of traffic accidents per passenger-car mile of travel in Massachusetts during 1953, classified by severity of accident.

accidents was $40.53 per passenger car registered.

Similar comparisons of costs per licensed passenger-car operator and per mile of road indicated a range of 88 cents to $15.44 and $67 to $1,171 for fatal- and nonfatal-injury accidents, respectively.

Figure 1 shows that the direct cost of operating a passenger car 1 mile was 0.17 cent for property-damage-only accidents, 0.25 cent for nonfatal-injury accidents, and 0.01 cent for fatal-injury accidents, or a total of 0.43 cent for each mile of passenger-car operation.

Type of Accidents

Motor-vehicle traffic accidents fall into one of five types: collision between motor vehicles, collision with pedestrians, collision with fixed objects, collision with other objects, and the noncollision-type accidents in which the vehicle turns over in the road or runs off the road.

It is evident in table 3 that of all accident types, collisions between motor vehicles were by far the most numerous. More than 8 out of 10 accidents involving passenger cars were of this type. Furthermore, out of a total of 1,131 accidents per 100 million vehicle-miles of travel, 943 involved passenger-car collisions with other motor vehicles.

Second from the standpoint of numbers of accidents was passenger-car collisions with objects, fixed and otherwise. This type accounted for 1 out of 10 accidents. About 1 in every 20 accidents involved passenger cars and pedestrians, and less than 1 in 50 accidents were the noncollision type. However, the number of accidents alone is not a measure of the relative economic significance of accidents of different types; both the number and the severity of accidents must be considered.

By relating the number of accidents as shown in table 3 with population, passenger cars registered, and licensed drivers, accident rates were as follows:

Table 4.—Direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by type of accident

Type of accident	Total direct cost	Percent of total	Total direct cost—					
			Per accident	Per capita	Per passenger car registered	Per licensed operator	Per mile of road	Per 100 million vehicle-miles of travel
	1,000 dollars							*1,000 dollars*
Passenger-car collision with—								
Other motor vehicles	41,816	83.3	$381	$8.76	$33.75	$22.50	$1,707	360
Pedestrians	3,375	6.7	572	.71	2.72	1.82	138	29
Fixed objects	3,023	6.0	414	.63	2.44	1.63	123	26
Other objects	673	1.3	105	.14	.54	.36	27	6
Noncollision accidents	1,337	2.7	608	.28	1.08	.72	55	11
All accidents	50,224	100.0	382	10.52	40.53	27.03	2,050	432

There was 1 collision between motor vehicles for every 44 persons, 1 collision with pedestrians for every 809 persons, 1 collision with fixed objects for every 654 persons, 1 collision with other objects for every 746 persons, and 1 noncollision accident for every 2,170 persons.

On the basis of passenger cars registered, there was 1 collision between motor vehicles for every 11 passenger cars, 1 collision with pedestrians for every 210 passenger cars, 1 collision with fixed objects for every 170 passenger cars, 1 collision with other objects for every 194 passenger cars, and 1 noncollision accident for every 563 passenger cars.

Considering the number of licensed operators, there was 1 collision between motor vehicles for every 17 drivers, 1 collision with pedestrians for every 315 drivers, 1 collision with fixed objects for every 255 drivers, 1 collision with other objects for every 290 drivers, and 1 noncollision accident for every 845 drivers.

The average direct cost for each type of accident, as shown in table 4, was found to be as follows: Collisions between motor vehicles, $381; collisions of passenger cars with pedestrians, $572; collisions with fixed objects, $414; collisions with other objects, $105; and noncollision accidents, $608.

These average costs reflect the severity of accidents of different types rather than their economic importance. It was found that noncollision accidents were the most severe type of accident, and following in order were passenger-car collisions with pedestrians, passenger-car collisions with fixed objects, collisions between motor vehicles, and passenger-car collisions with other objects.

The overriding economic impact of collisions between motor vehicles is made quite clear in table 4. Out of every accident direct-cost dollar, 83 cents applied to this type of accident. On the basis of 100 million vehicle-miles of travel, collisions between motor vehicles cost five times as much as all other types of accidents combined.

Other types of accidents ranked in order of cost (fractional parts of a dollar) were as follows: passenger cars colliding with pedestrians, 7 cents; collisions with fixed objects, 6 cents; noncollision accidents, 3 cents; and collisions with other than fixed objects, 1 cent.

By relating the direct cost data in table 4 to population, passenger cars registered, licensed operators, and road mileages, accident cost rates were determined as follows:

The per capita direct cost was $8.76 for collisions with other motor vehicles, 71 cents for collisions with pedestrians, 63 cents for collisions with fixed objects, 14 cents for collisions with other objects, and 28 cents for noncollision accidents.

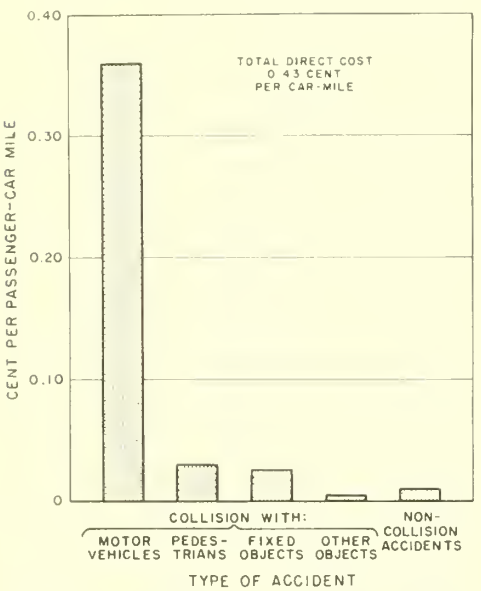

Figure 2.—Direct cost of traffic accidents per passenger-car mile of travel in Massachusetts during 1953, classified by type of accident.

Table 5.—Number of collisions between passenger cars or passenger cars and other motor vehicles in Massachusetts during 1953, classified by type of collision

Number and rate of collisions	Type of collision								All collisions
	Angle	Rear-end	Head-on	Side-swipe, same direction	Parking maneuver	Turning movement	Backing in traffic lane	Side-swipe, opposite direction	
Number of collisions	53,200	22,500	12,800	7,100	5,200	4,800	2,600	1,500	109,700
Percent of total	48.5	20.5	11.7	6.5	4.7	4.4	2.4	1.3	100.0
Number of collisions per 100 million vehicle-miles of travel	458	193	110	61	45	41	22	13	943
Number of persons per collision	90	212	373	[1] 225	------	------	------	------	44
Number of passenger cars registered per collision	23	55	97	[1] 58	------	------	------	------	11
Number of licensed operators per collision	35	83	145	[1] 88	------	------	------	------	17

[1] Includes "sideswipe, same direction" and the four remaining types of collisions.

Table 6.—Direct cost of collisions between passenger cars or passenger cars and other motor vehicles in Massachusetts during 1953, classified by type of collision

Type of collision	Total direct cost	Percent of total	Total direct cost—	
			Per collision	Per 100 million vehicle-miles of travel
	1,000 dollars			*1,000 dollars*
Angle	17,386	41.6	$327	150
Rear-end	10,842	25.9	482	93
Head-on	9,078	21.7	709	78
Sideswipe, same direction	1,958	4.7	276	17
Parking maneuver	599	1.4	115	5
Turning movement	1,114	2.7	232	10
Backing in traffic lane	133	.3	51	1
Sideswipe, opposite direction	706	1.7	471	6
All collisions	41,816	100.0	381	360

Table 7.—Number of accidents involving passenger cars (other than collisions between motor vehicles) in Massachusetts during 1953, classified by type of accident

Number and rate of accidents	Passenger cars colliding with—			Noncollision accidents	Total
	Pedestrians	Fixed objects	Other objects		
Number of accidents	5,900	7,300	6,400	2,200	21,800
Percent of total	27.1	33.5	29.3	10.1	100.0
Number of accidents per 100 million vehicle-miles of travel	51	63	55	19	188
Number of persons per accident	809	654	746	2,170	219
Number of registered passenger cars per accident	210	170	194	563	57
Number of licensed operators per accident	315	255	290	845	85

Table 8.—Direct cost of accidents involving passenger cars (other than collisions between motor vehicles) in Massachusetts during 1953, classified by type of accident

Type of accident	Total direct cost	Percent of total	Total direct cost—				
			Per accident	Per capita	Per passenger car registered	Per licensed operator	Per 100 million vehicle-miles of travel
Passenger-car collision with—	*1,000 dollars*						*1,000 dollars*
Pedestrians	3,375	40.1	$572	$0.71	$2.72	$1.82	29
Fixed objects	3,023	36.0	414	.63	2.44	1.63	26
Other objects	673	8.0	105	.14	.54	.36	6
Noncollision accidents	1,337	15.9	608	.28	1.08	.72	11
All accidents	8,408	100.0	386	1.76	6.78	4.53	72

The direct cost per passenger-car registered was $33.75 for passenger-car collisions with other motor vehicles, $2.72 for collisions with pedestrians, $2.44 for collisions with fixed objects, 54 cents for collisions with other objects, and $1.08 for noncollision accidents.

The direct cost per licensed passenger-car operator was $22.50 for passenger-car collisions with other motor vehicles, $1.82 for collisions with pedestrians, $1.63 for collisions with fixed objects, 36 cents for collisions with other objects, and 72 cents for noncollision accidents.

The direct cost per mile of road was $1,707 for passenger-car collisions with other motor vehicles, $138 for collisions with pedestrians, $123 for collisions with fixed objects, $27 for collisions with other objects, and $55 for noncollision accidents.

Figure 2 shows that the cost of operating a passenger car 1 mile was 0.36 cent for collisions between motor vehicles, 0.029 cent for passenger-car collisions with pedestrians, 0.026 cent for passenger-car collisions with fixed objects, 0.006 cent for passenger-car collisions with other objects, 0.011 cent for noncollision accidents, and a total accident direct cost of 0.43 cent per mile.

Collisions Between Motor Vehicles

All collisions between motor vehicles fall into one of the eight collision types as shown in table 5. The different types of collisions are listed in the order of their numerical importance. Angle collisions, which were by far the most numerous, accounted for nearly half of the collisions between motor vehicles. About 1 out of 5 collisions was a rear-end collision, and one in 9 was a head-on collision. These three collision types accounted for 80 percent of all collisions between motor vehicles.

There were less than half as many rear-end collisions and approximately one-fourth as many head-on collisions as there were angle collisions per 100 million vehicle-miles of travel. On this same basis there were more rear-end collisions than there were collisions of the following five types combined: sideswipes in the same direction, parking-maneuver collisions, turning-movement collisions, backing-in-traffic-lane collisions, and sideswipes in the opposite direction. In fact, these five types together accounted for only one-fifth of all collisions between motor vehicles.

Collision rates based on travel, population, registered vehicles, and licensed operators further emphasize the high frequencies of the angle type of collision.

It is evident from table 6 that angle collisions ranked far above all other types in economic importance. Almost 42 cents of the direct-cost dollar spent for collisions between motor vehicles resulted from angle collisions. The rear-end collision ranked second and accounted for almost 26 cents of every dollar spent for collision accidents. Following closely were head-on collisions, which accounted for almost 22 cents of every collision-cost dollar. The five remaining types of collisions were of considerably less economic importance. Together they accounted for less than 11 cents of every dollar.

A comparison of the average direct cost per collision reveals that the head-on type of collision was by far the most expensive of all types, and was followed in order by rear-end, sideswipe (opposite direction), and angle collisions.

The direct cost of operating a passenger car 1 mile, as shown in figure 3, was 0.15 cent for angle collisions, 0.09 cent for rear-end collisions, 0.08 cent for head-on collisions, and 0.04 cent for all other types of collisions. The total direct cost of all types of collisions between motor vehicles was 0.36 cent per mile of passenger-car operation.

Accidents, Excluding Collisions Between Motor Vehicles

Passenger-car accidents other than collisions between motor vehicles are ordinarily classified into eight types as follows: collisions with fixed objects, collisions with pedestrians, collisions with bicycles, collisions with animals or animal-drawn vehicles, collisions with railroad trains, collisions with streetcars, collisions with other objects, and noncollision accidents. However, since accidents involving collisions with bicycles, animals or animal-drawn vehicles, trains, and streetcars together accounted for less than 3,000 of the almost 22,000 passenger-car accidents, excluding collisions between motor vehicles, these four accident types were combined with other objects in tables 7 and 8.

Of the 21,800 accidents other than collisions between motor vehicles, 1 out of 10 was a

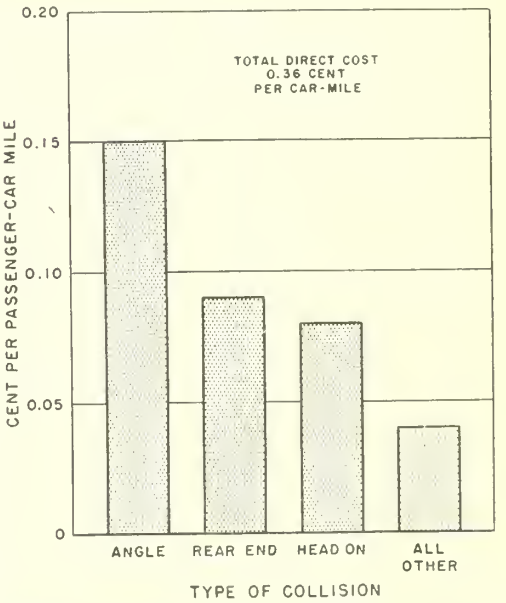

Figure 3.—Direct cost of collisions between passenger cars or passenger cars and other motor vehicles per passenger-car mile of travel in Massachusetts during 1953, classified by type of collision.

noncollision type in which the vehicle turned over in the road or ran off the road without striking anything. In 9 out of 10 accidents, the passenger car struck something other than another motor vehicle.

The costs shown in table 8 represent 16.6 percent of the total direct cost of all motor-vehicle traffic accidents experienced by Massachusetts passenger-car drivers during 1953. Not included in the table are collisions between motor vehicles.

Among the four types of accidents shown, collisions with pedestrians were of the greatest economic importance. Collisions with fixed objects ranked second, noncollision accidents ranked third, and collisions with other objects ranked last. On the basis of cost per accident, the noncollision-type accident ranked first and was followed by collisions with pedestrians, collisions with fixed objects, and collisions with other objects.

Direct Costs Summarized

The severity-class dollar illustrated in figure 4 is representative of the $50,224,000 spent on accidents in Massachusetts during 1953. This diagram shows that the cost of nonfatal-injury accidents is greater than that of fatal accidents and property-damage-only accidents combined. The diagram also portrays the minor economic role of fatal accidents.

The accident-type dollar, as diagramed in figure 4, brings accidents of different types into proper economic perspective. It illustrates the overriding economic importance of collisions between motor vehicles.

Figure 4 also illustrates the allocation of costs for the various types of collisions between motor vehicles. Angle, rear-end, and head-on collisions accounted for 90 cents of the collision-between-motor-vehicles dollar.

The noncollision and collision-with-objects dollar, as diagramed in figure 4, is the equivalent of a 17-cent segment of the accident-type dollar. It is representative of the $8,408,000 spent for noncollision accidents and passenger-car collisions with objects other than another motor vehicle.

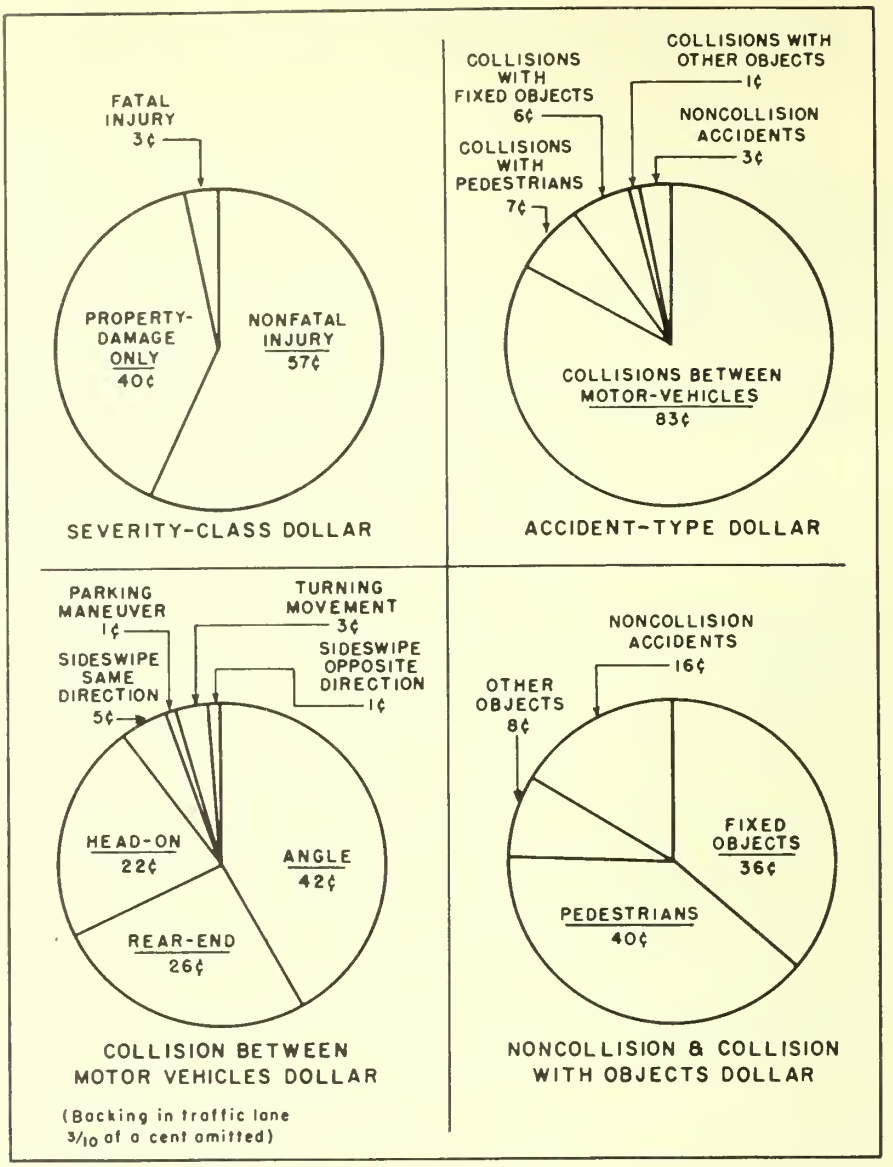

Figure 4.—Direct cost of passenger-car accidents shown as fractional parts of a dollar and classified according to severity of accident, type of accident, type of motor-vehicle collision, and other types of collision and noncollision accidents.

(Continued from page 37)

weighted average truck speed was entered in figure 6 to determine from the curve the truck equivalent in terms of passenger cars. Capacity analysis then was completed as previously described.

Descriptions of working procedures and the application of these data in estimating the requirements for truck lanes or other design modifications are discussed also in the manuals (*7, 8*) published by the State highway departments of Kentucky and Tennessee.

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Use of the Swiss Hammer for Estimating the Compressive Strength of Hardened Concrete

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB,
Highway Physical Research Engineer

A simple and portable instrument for use in estimating the compressive strength of hardened concrete in place has been developed recently. The device, popularly known as the Swiss Hammer, is designed for field use and is not intended as a substitute for control testing. It is being used in the field to gage increases in concrete strength with age and in locating low-strength areas when laboratory tests of control concrete cylinders or other conditions indicate that such areas might exist. It is also useful in surveys of old structures. The results of the tests given in this article show that factors such as surface smoothness, surface moisture condition, and type of coarse aggregate affect the strength values obtained by the use of the device.

A TEST METHOD which is simple, quick, and nondestructive has been developed by Ernest Schmidt, a Swiss engineer, for estimating the compressive strength of hardened concrete in place. The device consists of a steel plunger or hammer, free to travel in a tubular frame. When the head of the hammer is pressed against the surface of concrete, the hammer is retracted into the frame against the force of a tension spring. When the head is completely retracted, the spring is automatically released driving the hammer against the concrete. A small sliding pointer indicates the rebound of the hammer on a graduated scale. The scale is 75 mm. in length, and reads from zero to 100 in equally spaced divisions. The amount of this rebound, R, was found by the inventor to be related to the compressive strength of concrete.

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

A number of research organizations have studied the performance of the Swiss Hammer both in the laboratory and in the field. The consensus of their reports is that the empirical relationships between hammer rebound and strength are affected by moisture conditions of the concrete and type of aggregate, thus limiting the usefulness of the hammer to cases where an approximation of strength is all that is required. However, these reports do not include sufficient data to determine fully the capabilities of this instrument.

Testing Procedure

The surface of the concrete selected for test should be smooth and free from any rough spots or honeycomb. A surface produced by form work or troweling is usually satisfactory. When necessary, a smooth surface may be prepared by rubbing with a carborundum stone an area approximately 2 inches in diameter. A suitable stone is furnished in the carrying case of the apparatus.

In performing the test, the hammer is held perpendicular to the surface of the concrete and pressed against it until the hammer is released and strikes the surface of the concrete. While the device is still pressed firmly against the concrete, a button on the side of the instrument is pressed which locks the pointer in position. This permits the removal of the device to facilitate reading the amount of rebound. The apparatus is shown in figure 1.

For any selected area, five or more rebound readings are taken and the average of these readings is used to estimate the compressive strength. Areas where the reinforcing steel is known to be close to the surface, or where the coarse aggregate is exposed, are avoided.

The manufacturer of the instrument furnished a graph showing the relation between the compressive strength of the concrete and the rebound readings. This graph has been reproduced as figure 2. The data for establishing the relation represented by the curve were based on tests by the Swiss Federal Testing Laboratory. The curve for estimating the compressive strength shows values of rebound obtained when the hammer is held in a horizontal position against a vertical concrete surface. For other than horizontal positions of the hammer, a correction factor should be applied to the rebound readings before using the curve for estimating the strength of the concrete. A chart giving these correction factors was furnished by the manufacturer. These factors vary with the angle from the horizontal and the amount of the rebound; as the rebound reading increases, the correction factor decreases. For example with a rebound reading of 30, the corrections applied are as follows:

Angle from horizontal	*Correction factor*
(upward)	
90°	−6
60°	−5
30°	−3
0	0
(downward)	
30°	+2
60°	+3
90°	+4

Laboratory Tests

To determine the value of the Swiss Hammer as a tool for use in estimating the strength of concrete used in highway construction, three series of laboratory tests were made as well as numerous associated studies.

Series 1

The specimens used in this series were 6- by 12-inch cylinders submitted from various field projects. The concretes covered a wide variation in mixes and materials. All tests were made on specimens in a moist condition. Rebound readings were taken on the sides of the cylinders just prior to tests for compressive strength. The cylinders were tested in a vertical position with the side of the cylinder resting against an 8- by 12-inch machined-steel plate which in turn was supported by a wall of the laboratory. The hammer was held horizontal and perpendicular to the side

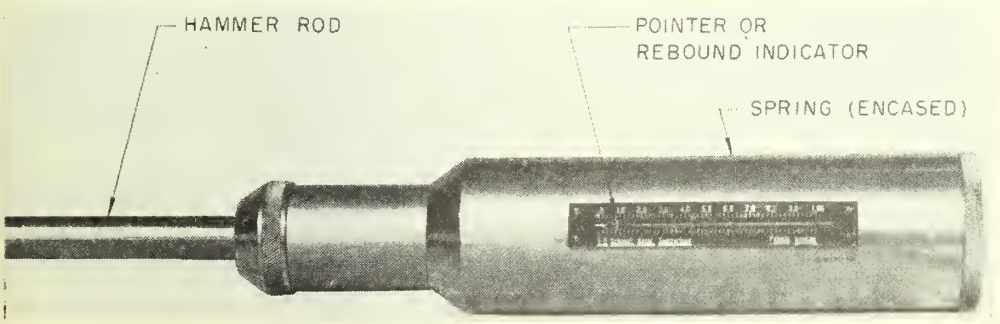

Figure 1.—Swiss rebound hammer.

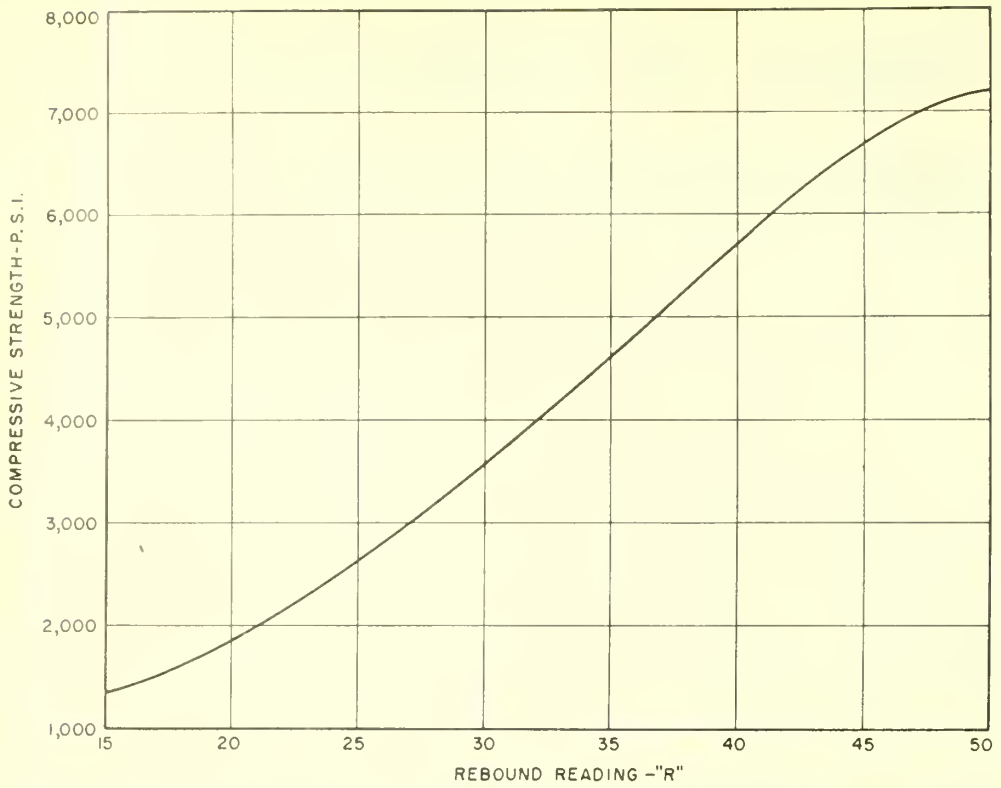

Figure 2.—Relation between compressive strengths and rebound readings as determined by the manufacturer.

of the cylinder. Usually 12 readings were taken on the side of each cylinder—3 readings on each quadrant with one reading 1 inch from the top, one at the center, and the other 1 inch from the bottom. Immediately after the rebound readings were taken, the cylinders were tested for compressive strength in a 400,000-pound hydraulic testing machine.

The results of the impact hammer and compressive strength tests on these cylinders are shown in figure 3. The upper curve represents the average relation between the rebound readings and the actual compressive strength. The strengths as shown by this curve are approximately 50 percent higher than the compressive strengths corresponding to the same rebound readings as shown in the curve furnished with the hammer. For example, the compressive strength for a rebound reading of 20, as determined from the curve for this series of tests, would be 2,750 p.s.i. as compared with 1,850 p.s.i. from the manufacturer's curve. For a rebound reading of 30, the compressive strength from the series 1 tests would be 5,300 p.s.i. as compared with only 3,600 p.s.i. from the manufacturer's curve for the same rebound reading.

The results of these tests indicated that the concrete cylinders held in the manner described did not have enough mass or rigidity to give reliable rebound readings, and that some of the energy from the blow may have been absorbed by movement of the cylinders.

Series 2

A second series of tests was made on another group of 6- by 12-inch cylinders submitted from projects under construction. To hold the cylinder firmly while the readings were taken with the hammer, each cylinder was put in the compression testing machine and a small load applied. A load of approximately 300 p.s.i. was found sufficient. Tests showed that greater loads had no effect on the rebound readings. After the rebound readings were taken, the cylinders were tested for compressive strength.

The results of these tests are shown in figure 4. The compressive strength for any rebound reading, as determined from the upper curve in figure 4, is approximately 12 percent higher than the compressive strength based on the curve submitted by the manufacturer of the hammer. Figure 5 shows the Swiss Hammer being used in the series 2 laboratory tests.

Series 3

In series 3 tests, the effect of type of coarse aggregate on the rebound-compressive strength relation was studied. All of the concrete cylinders were made in the laboratory and were tested as described in series 2. In the first part of the series (series 3A), four different gravels were used in making the cylinders tested. The results of the tests on these specimens are shown in figure 6. The spread in compressive strength among the curves representing the concrete prepared with the four gravel coarse aggregates varied from 250 to 600 p.s.i.

In the second part of this series, comparisons were made between concrete prepared with a siliceous gravel and crushed limestone. The curves giving the average relation between rebound readings and compressive strength for concrete containing these aggregates are shown in figure 7. The curve for the concrete prepared with crushed stone aggregate indicated about 25 percent greater strength for a given rebound reading than for the concrete prepared with gravel.

The curve for the relation between rebound readings and compressive strength for the gravel concrete corresponds very closely to that furnished by the manufacturer and shown in figure 2.

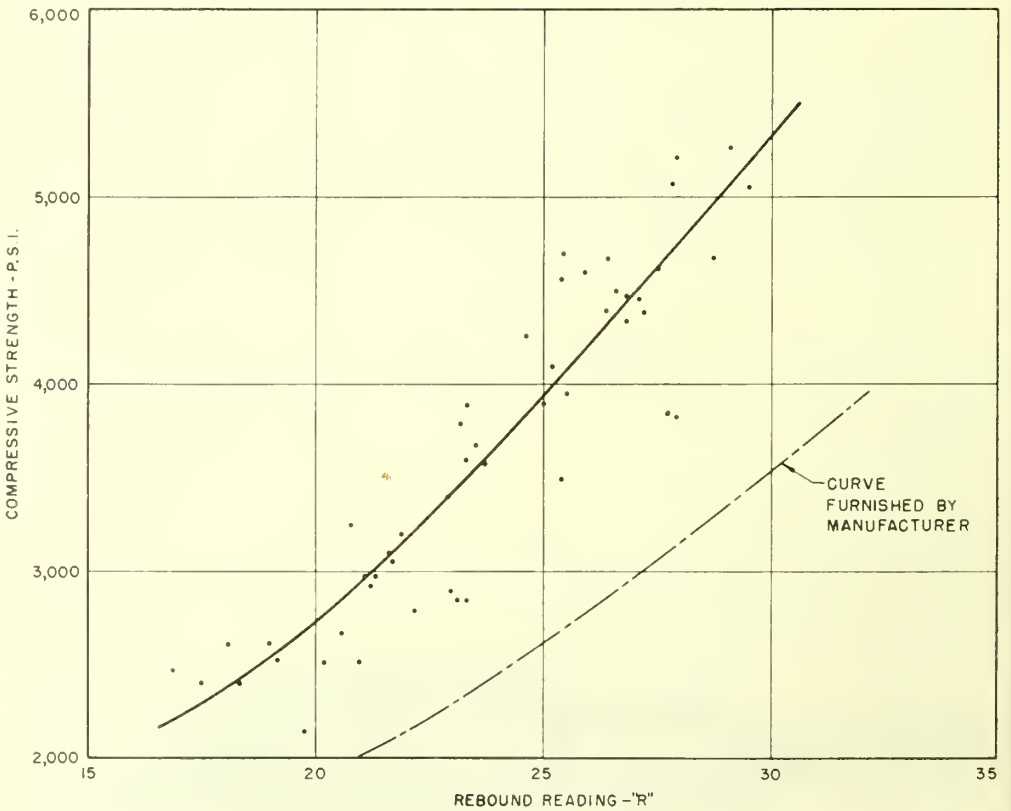

Figure 3.—Relation between compressive strengths and rebound readings on 6- by 12-inch concrete cylinders—series 1 tests.

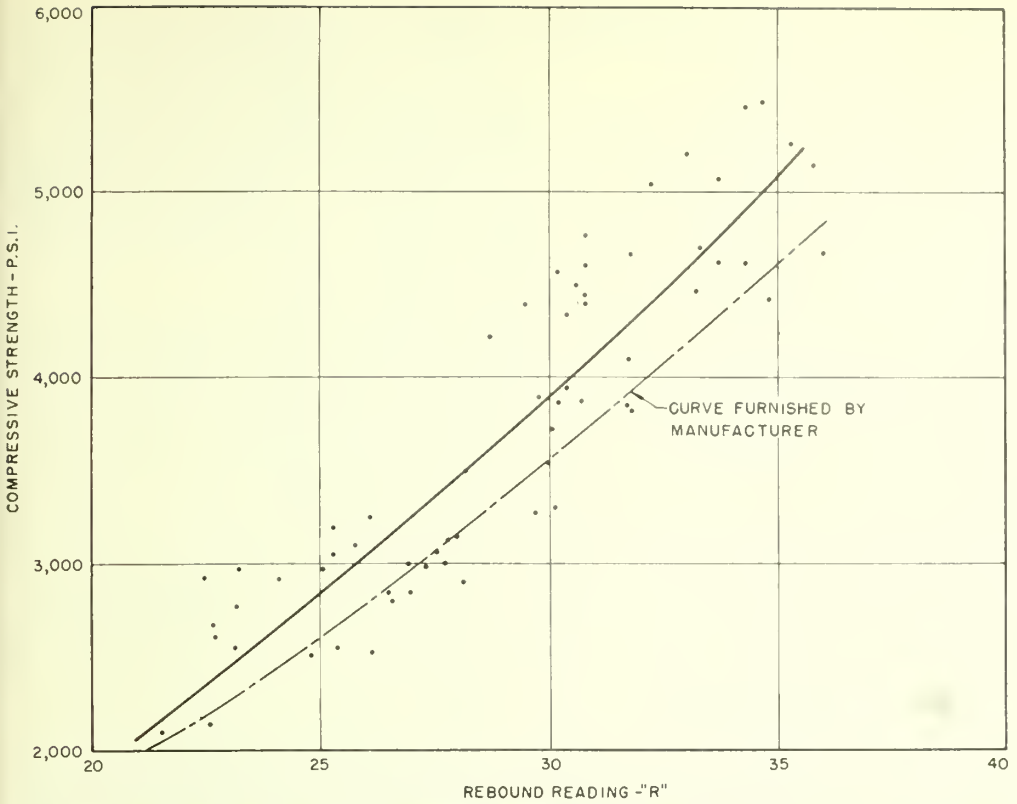

Figure 4.—Relation between compressive strengths and rebound readings on 6- by 12-inch concrete cylinders—series 2 tests.

These tests show that type of coarse aggregate is a governing factor in the rebound-compressive strength relation. This means that the Swiss Hammer is of most value in making comparative tests on concrete prepared with the same coarse aggregate. If comparisons between concretes prepared with different aggregates are desired, curves for the rebound-compressive strength relation for each aggregate should be obtained.

Associated Tests

Rebound readings were taken on the top and bottom of cylinders as cast prior to capping as well as on the sides. The readings were taken as described in the series 1 tests. There was considerable difference in the readings on the top, bottom, and sides of the same cylinder. The results are shown in table 1. This table also shows the estimated compressive strengths, which correspond to these readings, taken from the curve furnished by the manufacturer and the actual compressive strength of the concrete.

The average of the readings taken on the bottoms of all of the cylinders was 23 percent higher than the average of the readings taken on the sides of the cylinders, whereas the average of the readings on the top was only 5 percent higher. An explanation for some of this difference could be a difference between the quality of the concrete in the top and bottom of the cylinder. It is also possible that the cylinders were in a more rigid position when the readings were taken on the top and bottom than they were when readings were taken on the side.

Rebound readings were taken on a few cylinders in a dry condition and then on the same cylinders after immersion in water for 24 hours. The readings on the cylinders in a dry condition in all cases were larger than those in a moist condition. The results of these tests are shown in table 2. The estimated compressive strengths taken from the manufacturer's curve are also shown in this table.

A study was made to determine whether the rebound readings increased with age as the compressive strength increased. Of 12 concrete cylinders made from the same batch of concrete, 4 were tested at an age of 5 days, 4 at 10 days, and 4 at 20 days. The results of these tests are shown in table 3. The estimated compressive strength and the actual compressive strength of this concrete are also shown. The increase in rebound values was approximately proportional to the increase in the actual compressive strength.

Studies were made of the uniformity of the concrete in 6- by 6- by 21-inch beams, and

Figure 5.—The Swiss Hammer as used in the series 2 tests.

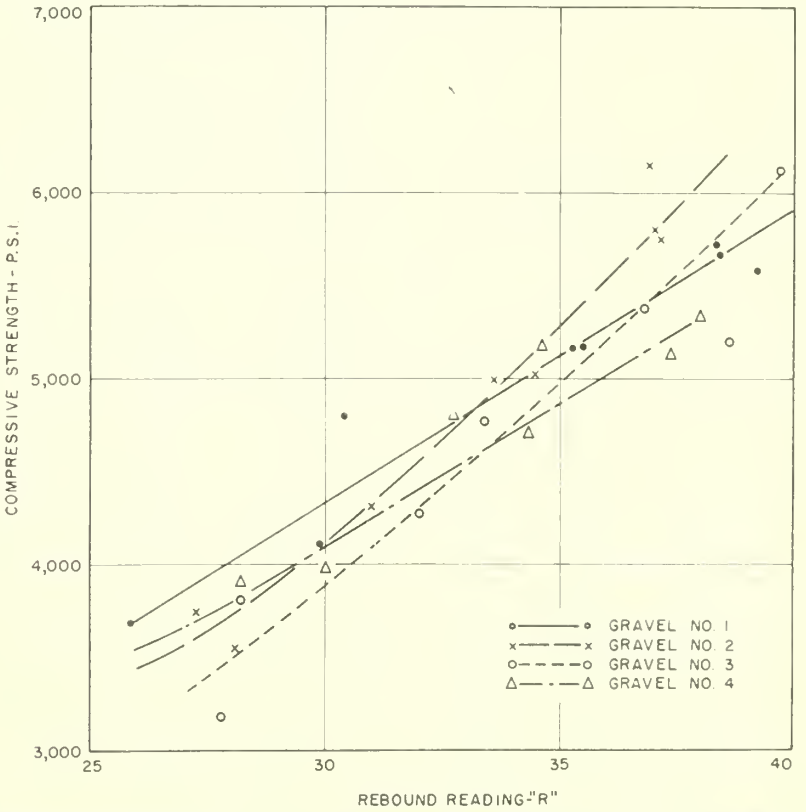

Figure 6.—Effect of gravel from different sources on rebound readings of 6- by 12-inch concrete cylinders—series 3A tests.

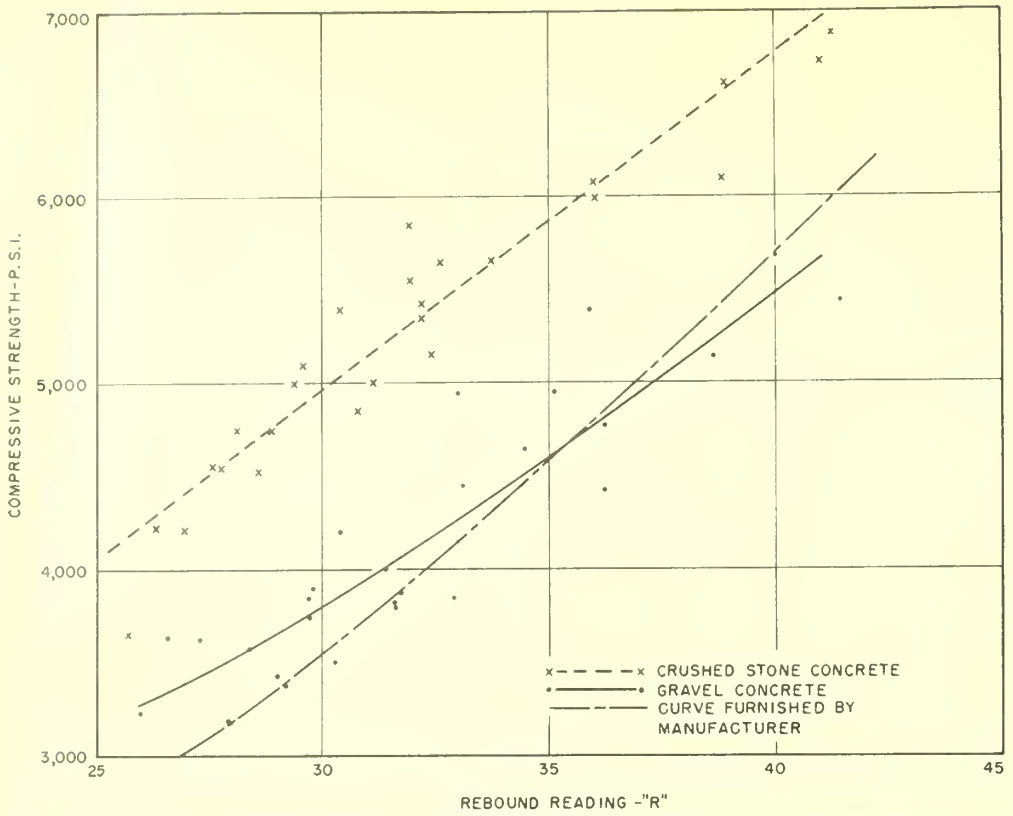

Figure 7.—Effect of type of coarse aggregate on rebound readings of 6- by 12-inch concrete cylinders—series 3B tests.

rebound readings were made on the sides, top, bottom, and ends of 29 beams. Five tests were made on the ends of each beam and 10 tests on each of the other faces. Average values for the entire group of beams were as follows:

Face of beam	*Rebound value*
Side	25. 5
Top	23. 6
Bottom	26. 1
End	28. 2

It is believed that these values correctly reflect slight differences between the quality of the concrete in different faces of the beams. With consideration given to the tendency of concrete to "bleed," the bottom of a beam should be more dense and have a higher rebound reading than the sides or the top. The rebound tests at the ends of the beams were made on a concrete specimen with a depth of 21 inches—3½ times the depth of concrete at any other point. This may be the reason for the greater readings.

A study was also made of the relation between rebound readings taken on the ends of 6- by 6- by 21-inch beams and readings taken on the sides of 6- by 12-inch cylinders. A beam and a corresponding cylinder were made from the same batch of concrete. Half of the total number of specimens contained gravel coarse aggregate and the other half contained crushed stone. The readings were taken on the beams held against the wall and the cylinders were placed in the testing machine with a small applied load as described in the series 2 tests. The beams were tested for flexural strength after the rebound readings were made.

The rebound values are shown in table 4 for both beams and cylinders, together with the actual compressive strengths of the cylinders and the flexural strengths of the beams. The average of all rebound readings on the cylinders for gravel concrete was 31.2 as compared with 30.9 for the beams made with gravel concrete. The average rebound reading was 32.3 for both cylinders and beams made from stone concrete.

There appears to be a definite relation between rebound readings taken on the ends of the beams and the flexural strength for this series of tests. This relation is shown in figure 8.

The Swiss Hammer could be used to estimate the flexural strength of paving concrete. Readings taken on control beams would indicate increases in flexural strength with age. From these readings, the age at which flexural strength tests should be made to meet specification requirements may be determined. This would reduce the number of control beams necessary.

Table 1.—Rebound readings on top, bottom, and side of 6- by 12-inch concrete cylinders

Cylinder number	Average rebound reading on side [1]	Estimated compressive strength [2]	Average rebound reading on top [1]	Estimated compressive strength [2]	Average rebound reading on bottom [1]	Estimated compressive strength [2]	Actual compressive strength [3]
		P. s. i.		*P. s. i.*		*P. s. i.*	*P. s. i.*
1	18. 3	1, 640	20. 8	1, 960	24. 2	2, 480	2, 410
2	19. 2	1, 750	22. 2	2, 160	23. 8	2, 420	3, 210
3	20. 8	1, 960	21. 8	2, 100	27. 4	3, 050	3, 250
4	21. 3	2, 040	21. 6	2, 050	29. 4	3, 420	2, 980
5	21. 3	2, 040	22. 0	2, 130	25. 4	2, 690	3, 290
6	22. 2	2, 160	22. 0	2, 130	29. 2	3, 380	2, 800
7	22. 8	2, 250	24. 3	2, 500	26. 9	2, 960	3, 080
8	23. 0	2, 280	23. 5	2, 360	27. 4	3, 050	2, 950
9	23. 5	2, 360	24. 9	2, 600	28. 0	3, 160	3, 270
10	25. 6	2, 730	26. 0	2, 800	30. 6	3, 650	3, 900
11	26. 9	2, 960	31. 0	3, 730	34. 8	4, 560	3, 870
12	26. 9	2, 960	26. 5	2, 890	30. 2	3, 570	4, 180
13	27. 9	3, 140	30. 0	3, 530	32. 8	4, 120	3, 800
14	28. 0	3, 160	27. 0	2, 980	33. 8	4, 340	4, 790
15	28. 7	3, 290	29. 2	3, 380	33. 8	4, 340	4, 700
16	29. 4	3, 420	32. 2	3, 980	35. 8	4, 780	4, 200
Average	24. 1	2, 510	25. 3	2, 700	29. 6	3, 500	3, 540
Index	100	----------	105	----------	123	----------	----------

[1] Each rebound value on side is an average of 12 readings; values on top and bottom are averages of 5 readings.
[2] Estimated values taken from curve in figure 2.
[3] Results of strength tests on cylinders.

Field Tests

The Swiss Hammer was used to estimate the strength of several concrete structures in the field. In one case, tests were made on three beam sections cast for post tensioning for use in a concrete bridge. Readings were taken on all beams prior to stressing and on one beam after stressing.

The concrete used in the beams was made with gravel aggregate. Each beam measured approximately 3 feet by 3 feet by 75 feet. Rebound readings were taken along the length of the beam at intervals of about 3 feet from one end to the center. Three readings were taken at each location: one reading 5 to 10 inches from the top, one at the center, and the other about 10 inches from the bottom. The beams were cured with wet burlap on the job site and were in a moist condition when readings were taken. Figure 9 shows the hammer being used on these beams.

Concrete cylinders for control were cast at the same time the beams were made. These were stored on the job for 5 days and then taken to a laboratory for moist storage and testing.

The rebound readings on the beams, the estimated compressive strength of the concrete in each beam as obtained from the curve furnished by the manufacturer (fig. 2), and the actual strengths of the test cylinders are shown in table 5. The average estimated compressive strengths of the beams were approximately 20 percent greater than the average compressive strengths of the test cylinders. Differences in curing and testing

Table 2.—Rebound readings on concrete cylinders in dry condition and after 24 hours immersion in water [1]

Cylinder number	Average rebound reading, dry	Estimated compressive strength [2]	Average rebound reading, wet	Estimated compressive strength [2]
		P. s. i.		*P. s. i.*
1	26.8	2,940	25.0	2,620
2	27.7	3,100	27.3	3,040
3	27.8	3,120	26.9	2,960
4	28.3	3,220	26.0	2,800
5	28.7	3,280	26.9	2,960
6	29.4	3,420	26.3	2,860
7	34.7	4,540	30.5	3,630
8	35.1	4,620	32.8	4,120
9	35.8	4,780	32.5	4,050
10	37.6	5,170	34.4	4,470
Average..	31.2	3,820	28.9	3,350

[1] Each value is an average of 12 readings. Approximate age of dry cylinders was 14 days.
[2] Estimated values taken from curve in figure 2.

procedures or in materials used may account for this variation.

With few exceptions the individual rebound readings show very little variation from the average. A few readings were excluded because the hammer had probably been held against a piece of exposed aggregate or against a thin layer of mortar over a void.

The Swiss Hammer was also used on the piers of a bridge which were about 2½ years old. The average compressive strength of control test cylinders at the age of 28 days was reported as 4,500 p.s.i. The average estimated compressive strength of this concrete at 2½ years, as determined from the rebound readings given in table 6 and the curve in figure 2, was 5,660 p.s.i.

The reconstruction of a bridge which was about 38 years old offered an excellent opportunity to try the Swiss Hammer on concrete in place and on specimens obtained for tests in the laboratory. This concrete was in good condition. Swiss Hammer readings were taken at four locations on the vertical face of the hand rail of the bridge. The average rebound reading of 40.9, when checked against the curve in figure 2, indicates a compressive strength of 5,850 p.s.i.

Three prisms approximately 6 inches square and 9 inches high were sawed from the hand rail at about the same location where the rebound readings were taken. After these specimens had been prepared for tests of compressive strength, they were placed under a small load in the testing machine. Rebound readings of these specimens had an average value of 41.2, corresponding to an estimated compressive strength of 5,980 p.s.i. The actual compressive strength of the three prisms corrected for H/D (height over depth) averaged 5,470 p.s.i.

In two of the three trials of the Swiss Hammer on concrete structures, direct comparisons could be made between the actual compressive strength of test specimens representing the concrete and the compressive strength as determined from the rebound reading and use of the curve in figure 2. In both cases, the rebound reading indicated a strength 10 to 20 percent higher than was obtained by

Table 3.—Rebound readings on concrete cylinders tested at various ages

Age at test	Average rebound reading [1]	Estimated compressive strength [2]	Actual compressive strength [3]
Days		*P. s. i.*	*P. s. i.*
5	21.9	2,110	3,410
10	25.0	2,620	3,920
20	28.3	3,220	4,800

[1] Each rebound reading is an average of 12 readings on each of 4 cylinders.
[2] Estimated values taken from curve in figure 2.
[3] Results of strength tests on cylinders.

Table 4.—Relation between rebound readings on concrete cylinders and beams prepared with two types of aggregate [1]

Gravel concrete				Stone concrete			
Rebound reading		Actual compressive strength of cylinders	Modulus of rupture of beams	Rebound reading		Actual compressive strength of cylinders	Modulus of rupture of beams
Side of cylinders [2]	End of beams [2]			Side of cylinders [2]	End of beams [2]		
		P. s. i.	*P. s. i.*			*P. s. i.*	*P. s. i.*
24.6	25.5	2,920	475	24.4	26.2	4,020	675
25.3	23.5	2,380	450	25.7	25.1	3,660	670
26.0	25.5	3,220	455	26.3	27.2	4,220	710
26.6	27.0	3,640	445	27.1	26.0	4,230	735
26.8	25.2	2,450	490	27.6	28.0	4,560	720
27.3	27.3	3,630	460	27.8	28.8	4,550	740
27.9	28.6	3,190	510	28.1	28.8	4,760	720
28.4	26.5	3,570	470	28.6	28.8	4,520	785
29.0	27.3	3,420	420	28.9	28.2	4,740	755
29.2	28.7	3,380	480	29.4	29.6	4,990	690
29.7	28.8	3,740	460	29.6	28.4	5,090	720
29.7	30.7	3,840	470	30.4	32.6	5,400	725
29.8	29.7	3,900	440	30.8	30.4	4,850	800
30.3	31.1	3,490	480	31.1	29.3	4,990	795
30.4	30.5	4,200	445	31.9	31.4	5,540	735
31.4	30.6	3,990	480	31.9	33.6	5,840	770
31.6	30.6	3,820	480	32.2	32.0	5,360	780
31.6	31.8	3,790	560	32.2	34.4	5,430	840
31.7	30.8	3,880	535	32.4	31.9	5,140	815
32.9	30.1	3,860	595	32.6	31.8	5,650	690
33.0	35.0	4,950	515	33.7	35.3	5,660	930
33.1	33.4	4,450	610	36.0	37.3	6,010	955
34.5	33.5	4,640	575	36.0	37.6	6,080	905
35.2	35.1	4,960	490	38.8	37.5	6,100	775
35.9	37.0	5,400	590	38.8	38.8	6,330	925
36.2	40.0	4,780	675	39.1	36.9	6,620	845
36.2	33.8	4,420	610	40.9	40.2	6,740	925
38.6	39.4	5,140	595	41.2	39.6	6,890	915
41.4	40.5	5,440	650	43.7	41.9	7,090	925
(31.2)	(30.9)	(3,950)	(515)	(32.3)	(32.3)	(5,350)	(790)

[1] Figures in parentheses are average values for all tests.
[2] Each value is an average of 10 or 12 readings.

Table 5.—Rebound readings obtained with Swiss Hammer in field tests of beams for a post-tensioned bridge

Beam No. 5, age 8 days	Beam No. 5, age 16 days	Beam No. 2, age 35 days	Beam No. 4, age 21 days	Beam No. 4,[1] age 29 days
Rebound Readings [2]				
35	44	31	38	41
36	[3] 47	39	38	42
39	44	40	38	40
37	43	41	36	39
37	38	38	37	40
[3] 44	37	37	35	38
34	44	39	40	40
41	36	38	35	38
36	37	40	40	38
34	36	37	36	41
33	35	[3] 48	38	39
34	38	41	[3] 28	40
35	41	36	40	41
33	37	36	38	37
39	42	40	38	42
33	33	36	38	39
35	36	36	43	37
36	34	36	39	38
31	39	36	34	43
[3] 51	37	35	40	43
37	42	[3] 45	38	47
30	34	34	[3] 47	37
30	38	34	38	37
38	41	34	38	44
33	34	37	[3] 48	45
35	34	38	35	38
37	37	37	42	39
31	38	-------	-------	-------
36	36	-------	-------	-------
40	39	-------	-------	-------
(35.2)	(38.1)	(37.0)	(38.0)	(40.1)
Compressive Strength [4] (p. s. i.)				
(4,640)	(5,280)	(5,040)	(5,260)	(5,720)
3,700	4,100	4,600	4,600	4,700
[5] 25	[5] 29	[5] 10	[5] 14	[5] 22

[1] Readings taken after beam had been stressed.
[2] Figures in parentheses are averages of all rebound readings.
[3] Readings not included in averages.
[4] Figures in parentheses represent estimated compressive strengths of concrete, whereas figures immediately below are the actual compressive strengths of the control cylinders.
[5] Percentage that estimated compressive strength exceeded actual compressive strength of control cylinders.

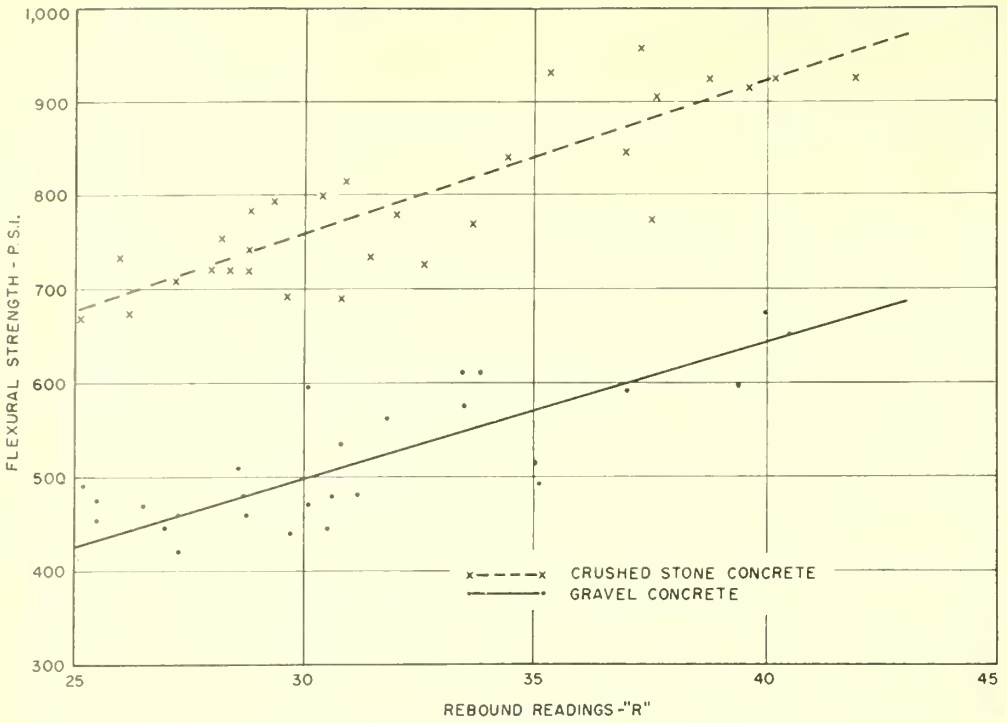

Figure 8.—Relation between flexural strengths and rebound readings on 6- by 6- by 21-inch concrete beams prepared with two types of aggregate.

testing the specimens. It is apparent that the curve given in figure 2 should be used with reservations. For the best determinations, a curve should be prepared showing the relation between strength and rebound reading for concrete of the same type and composition as that to be inspected.

Considerable wear was found on the face of the striking rod of the Swiss Hammer after making the tests described in this article. Check tests made in the laboratory showed little effect on the indicated reading from this wear. However, for extended use it would be desirable to have a harder wearing surface on the face of the hammer.

Factors Affecting Results of Tests Using Swiss Hammer

In using the Swiss Hammer, there are a number of factors which affect the readings. The following conditions should be considered in interpreting the results:

Condition of the surface of the concrete.—Readings taken on a polished surface are high, whereas readings taken on a rough surface (such as a broomed surface) are low.

Moisture condition on the concrete.—Concrete in a moist condition gives a lower reading than concrete in a dry condition.

Type of coarse aggregate.—The type of coarse aggregate used and possibly the composition of the concrete affect the amount of rebound.

Value of the Swiss Hammer

The Swiss Hammer provides a quick and inexpensive method for checking the uniformity and estimating the strength of hardened concrete. It is not intended as a substitute for control test cylinders, nor is it intended to give an accurate measure of the compressive strength of the concrete. It is valuable for use in the field for "trouble shooting" to determine whether test cores are needed and where they should be drilled. It may be used to determine the rate of increase in strength of concrete and to determine when forms can be removed or loads applied. It may also be used to estimate the extent of damage to structures caused by freezing or fire, and to judge the quality of the concrete in old structures.

Table 6.—Rebound readings obtained with a Swiss Hammer in field testing of bridge piers

Rebound readings [1] for—			
West pier	Center pier, east side	Center pier, west side	East pier
41	42	37	39
41	42	31	37
42	48	43	35
42	43	37	35
42	40	38	39
41	40	38	40
42	38	43	40
44	44	50	40
39	48	42	34
43	32	34	36
38	43	34	38
40	36	41	32
42	38	42	38
41	36	39	36
46	41	40	46
41	41	40	38

[1] The respective average rebound readings and estimated compressive strengths of concrete cylinders were as follows: west pier, 41.6 and 6,030 p. s. i.; center pier (east side), 40.8 and 5,870 p. s. i.; center pier (west side), 39.3 and 5,550 p. s. i.; and east pier, 37.7 and 5,200 p. s. i.

Figure 9.—Swiss Hammer being used in field testing.

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Wind Tunnel for Aerodynamic Testing of Section Models of Suspension Bridges

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

A WIND TUNNEL for aerodynamic testing of section models of suspension bridges has been constructed at the Bureau of Public Roads Research Station near Langley, Va. It was designed by personnel of the Bureau of Public Roads after consulting with officials of the Aeronautics Section of the National Bureau of Standards. Much of the operating equipment and instrumentation was also built in the Bureau's shops. A few instruments are yet to be provided.

The failure of the Tacoma Narrows Bridge on November 7, 1940, in a wind of moderate velocity made it evident to engineers concerned with the design of suspension bridges that it would be necessary to make a thorough investigation of the performance of bridges of this type when subjected to wind action, if future bridges were to be designed with assurance that they would be safe from catastrophic motion. Upon the request of a representative group of these engineers, the Commissioner of Public Roads organized the Advisory Board on the Investigation of Suspension Bridges for the purpose of initiating and correlating such investigations. Following the recommendation of the Advisory Board, the Bureau engaged in several investigations in cooperation with the Washington Toll Bridge Authority, the University of Washington, the Golden Gate Bridge and Highway District, the American Institute of Steel Construction, and the Oregon State Highway Department.

Previous Studies

The studies that were made in cooperation with the Washington Toll Bridge Authority and the University of Washington accomplished the objectives of determining quite satisfactorily the nature of the wind action which caused the failure of the original girder-stiffened Tacoma Narrows Bridge and indicating the design provisions needed to insure the stability of the new truss-stiffened structure which has replaced it. These studies also showed how a scaled section model, representing a limited length of a suspension bridge and mounted in the wind tunnel on properly designed springs, may be used to show the aerodynamic characteristics of the bridge without the necessity of testing a complete scale model.

The studies were extended to cover a number of other girder- and truss-stiffened bridges including the Golden Gate Bridge as originally built. The cooperative observations on the Golden Gate Bridge provided the opportunity to correlate the behavior of a bridge in the field and its model in the wind tunnel. Analytical studies by the late Dr. Friedrich Bleich,[1] made possible through the cooperation with the American Institute of Steel Construction, provided rational correlation with both the laboratory tests and the field observations.

Major design features which provide aerodynamic stability in the new Tacoma Narrows Bridge are (1) the bottom lateral system which materially increases the torsional stiffness of the structure and raises the frequency of any torsional oscillation which might occur, and (2) the grated slots in the roadway which break up the action of wind forces tending to excite "flutter," a form of coupled oscillation of bending and torsional motions. The studies have shown these design features to be effective in stabilizing other truss-stiffened suspension bridge sections but the nature and degree of the benefit varies. For this reason, tests and studies on a specific design are considered advisable in order to establish the effectiveness and optimum proportions of the stabilizing features. Brief studies on means for stabilizing girder-stiffened sections have not as yet yielded full solutions, and additional data on this factor are needed.

The wind tunnel at the University of Washington, which was specially designed for testing suspension bridge models, was of temporary construction and will not be available for continued studies. The Advisory Board on the Investigation of Suspension Bridges recommended that the Bureau of Public Roads provide facilities to continue the testing of section models. A nationwide survey indicated that it would cost less to provide and operate a special wind tunnel in the laboratory of the Bureau than to make the test-section adaptations and other modifications required to equip an ordinary wind tunnel for this special type of testing. The use of an ordinary wind tunnel would result in an intermittent test program because of the necessity of fitting the program into an already crowded working schedule.

Mr. George S. Vincent, who for 11 years represented the Bureau of Public Roads at the University of Washington while the cooperative wind-tunnel studies were being made there, will be in charge of the Bureau's research program on bridge section models. Mr. Vincent is a coauthor of the report *Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge,* which was published as a result of the studies made at the University of Washington.

[1] *The mathematical theory of vibration in suspension bridges,* by Bleich, McCullough, Rosecrans, and Vincent. Chapters 3-8. Bureau of Public Roads, 1950.

Description of Wind Tunnel

The type of testing required for bridge section models utilizes only relatively low wind velocities but demands fine control of the wind and more than ordinary precision in the measurement of wind velocity. Provision for varying the direction of the wind in the vertical plane is also required. The test section of the new tunnel is open and has a nozzle 6 feet square. Control of the vertical angle of the wind is accomplished by rotating the nozzle about the cylindrical pressure chamber. The model, mounted on springs designed to reproduce the proper scaled frequency of oscillation, can be moved vertically and longitudinally on its supporting mechanism in order to place it in the desired position in the windstream.

Wind velocities up to about 50 feet per second are provided by a 60-inch double inlet fan driven by a 50-horsepower direct current motor which can be controlled continuously from creep to a maximum speed of about 500 r. p. m. The airstream from the fan passes through a series of 40-mesh, stainless steel diffusing screens in a duct which diverges vertically and laterally and leads to a 13- by 13-foot pressure chamber, the size being restricted by the headroom available. This produces a very uniform flow of air at the nozzle. A double screen, pivoted at the center of the pressure chamber, can be rotated as may be required to correct the flow when the vertical angle of the windstream is varied. Finely controlled relief or bypass openings just downstream from the fan afford additional control of the wind velocity.

The features described are shown in figure 1. Figure 2 is a photograph showing a section model of the new Tacoma Narrows Bridge mounted on springs in front of the nozzle of the wind tunnel. The carriage for the model, rolling on the tracks which support its ends, can be moved longitudinally with respect to the axis of the windstream, and the entire assembly can be fixed at any desired elevation on the four supporting tubular columns. At the far left, on a concrete pedestal, can be seen one of the axle bearings on which the nozzle may be rotated vertically by raising or lowering its outlet end.

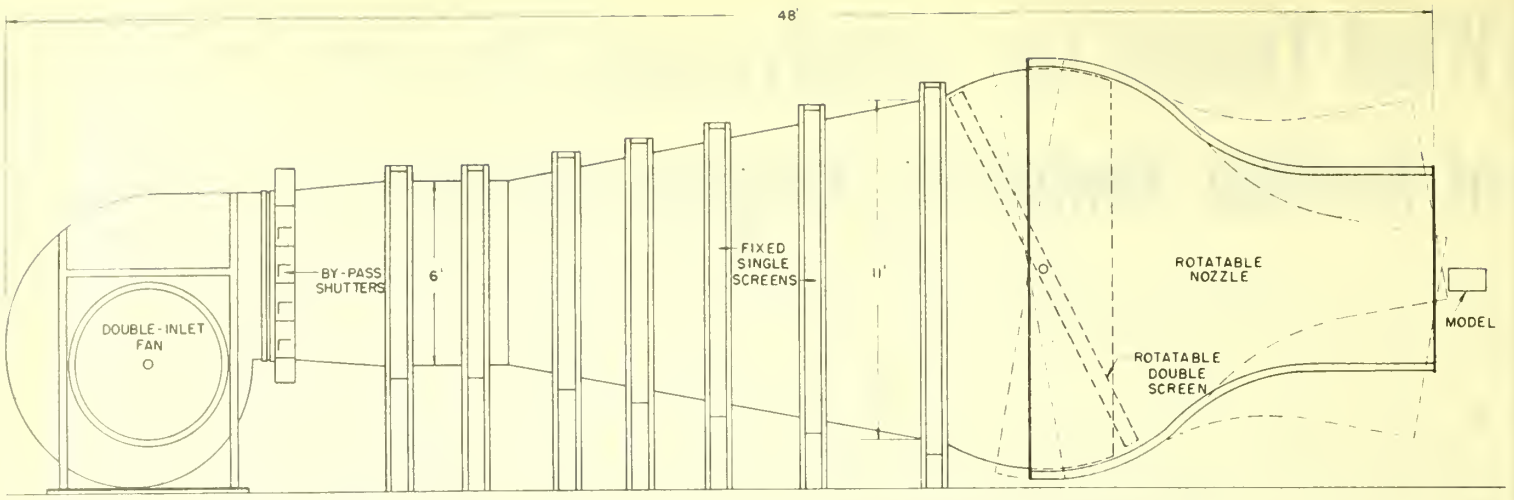

Figure 1.—Side elevation giving shape and principal dimensions of wind tunnel.

Figure 2 also shows two pitot-static tubes mounted at the sides of the nozzle. These can be moved to cover any part of the airstream at the nozzle and for some distance downstream. They are connected to the manometer at the left which is read by a micrometer-mounted telescope. The console in the foreground carries the controls for the fan motor and for the bypass shutter mechanism as well as meters indicating the rate of rotation of the fan, the amperage of the drive motor, and the amperage of the motor-generator converter. Space is also provided for other meters and instruments to be used in the tests.

Following tests of the characteristics and control of the windstream itself, the tunnel will be used to repeat tests on section models previously tested in the wind tunnel at the University of Washington in order to determine whether they are influenced by individual "tunnel effects" such as are commonly observed in aeronautical research. High on the list of subsequent testing will be studies to discover means for stabilizing girder-stiffened suspension bridges.

The wind tunnel can be used to study a variety of problems involving wind-excited vibration of structures such as overhead signs on expressways, for example. For tests requiring wind velocities exceeding 50 feet per second, the nozzle can be choked with false walls, provided the dimensions of the model permit reducing the height or width of the test section.

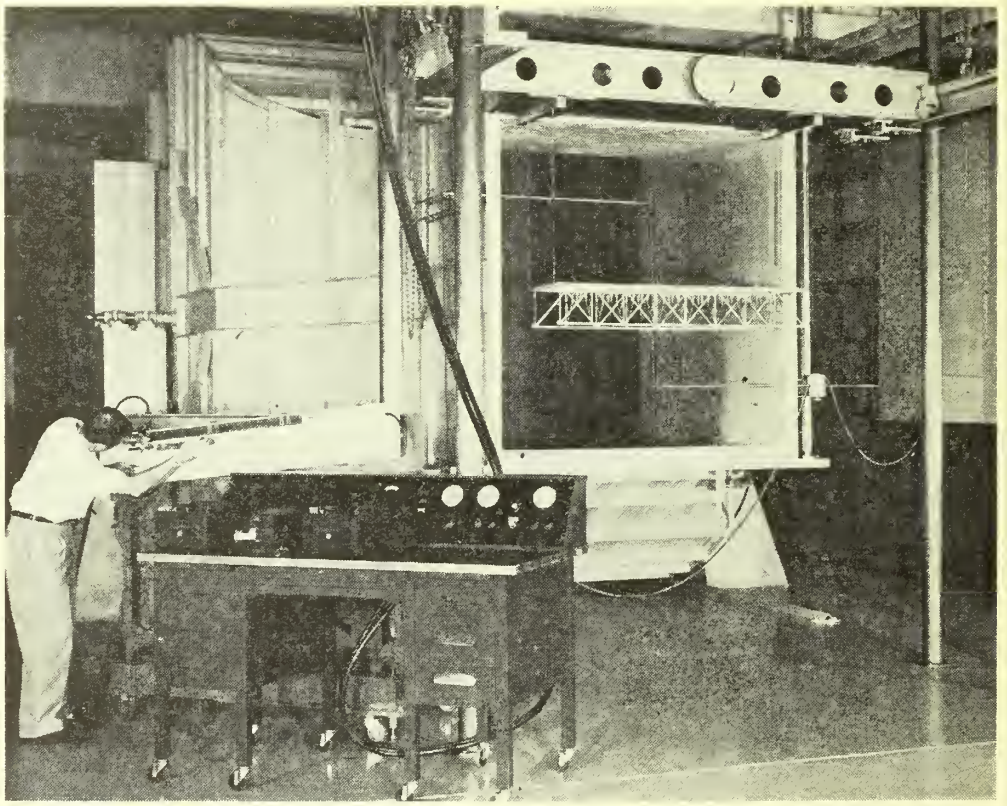

Figure 2.—Nozzle of wind tunnel with a bridge section mounted in front. The console in the foreground provides controls for the operation of the wind tunnel.

Power Shovel Productivity: A Motion Picture

The Bureau of Public Roads, U. S. Department of Commerce, recently announced the release of a new motion picture, *Power Shovel Productivity*. The film, based on extensive studies conducted by Public Roads, highlights the job conditions that determine the yardage output of power shovels on highway grading work, and demonstrates how production is affected by the speed of dipper cycle, size of dipper load, and frequency and duration of minor delays.

The motion picture is a 16-mm. sound and color film with a running time of 30 minutes. Prints may be borrowed for showings by any responsible organization by request addressed to Visual Education, Bureau of Public Roads, Washington 25, D. C. There is no charge except for the express or postage fees. At the present time, bookings are solid through the summer months. Requests for the film should be sent well in advance of the desired showing and alternate dates for showing should be given if possible. Immediate return after each showing is necessary, so that all requested bookings may be fulfilled.

Prints of the film may be purchased at $111.88 per copy, the price including film, reel, can, and shipping container, and postage within the United States. Inquiries should be addressed to Visual Education, Bureau of Public Roads, Washington 25, D. C. Payment should *not* be sent with the inquiry.

U. S. GOVERNMENT PRINTING OFFICE: 1958

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1951, 35 cents. 1954 (out of print). 1957, 30 cents.
1952, 25 cents. 1955, 25 cents.

PUBLICATIONS

A Report of Factors for Use in Apportioning Funds for the National System of Interstate and Defense Highways, House Document No. 300 (1958). 15 cents.

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). Out of print.

Consideration for Reimbursement for Certain Highways on the Interstate System, House Document No. 301 (1958). 15 cents.

Construction of Private Driveways, No. 272MP (1937). 15 cents.

Criteria for Prestressed Concrete Bridges (1954). 15 cents.

Design Capacity Charts for Signalized Street and Highway Intersections (reprint from PUBLIC ROADS, Feb. 1951). 25 cents.

Electrical Equipment on Movable Bridges, No. 265T (1931). 40 cents.

Factual Discussion of Motortruck Operation, Regulation, and Taxation (1951). 30 cents.

Financing of Highways by Counties and Local Rural Governments: 1931–41, 45 cents; 1942–51, 75 cents.

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General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Capacity Manual (1950). $1.00.

Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.

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1946 (out of print). 1950 (out of print). 1954, 75 cents.
1947 (out of print). 1951, 60 cents. 1955, $1.00.
1948, 65 cents. 1952, 75 cents.

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Legal Aspects of Controlling Highway Access (1945). 15 cents.

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Manual on Uniform Traffic Control Devices for Streets and Highways (1948) *(including 1954 revisions supplement)*. $1.25.
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Needs of the Highway Systems, 1955 84, House Document No. 120 (1955). 15 cents.

Opportunities in the Bureau of Public Roads for Young Engineers (1958). 20 cents.

Parking Guide for Cities (1956). 55 cents.

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Standard Plans for Highway Bridge Superstructures (1956). $1.75.

Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.

Transition Curves for Highways (1940). $1.75.

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Bibliography on Automobile Parking in the United States (1946).
Bibliography on Highway Lighting (1937).
Bibliography on Highway Safety (1938).
Bibliography on Land Acquisition for Public Roads (1947).
Bibliography on Roadside Control (1949).
Express Highways in the United States: a Bibliography (1945).
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U.S. DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1958

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM											
		PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$33,229	$58,583	$47,592	345.0	$20,920	$13,543	150.9	$87,658	$57,669	929.8	$167,161	$118,804	1,425.7
Arizona	14,918	26,367	23,386	165.4	7,515	6,929	48.2	39,095	33,928	203.1	72,977	64,243	416.7
Arkansas	27,294	44,221	35,452	522.6	20,401	15,235	133.5	49,691	35,707	414.3	114,313	86,394	1,070.4
California	66,483	52,836	30,964	218.3	29,514	24,178	32.7	589,436	273,631	424.9	671,786	328,773	675.9
Colorado	44,557	12,035	9,048	92.5	2,568	2,345	4.2	62,150	44,131	347.3	76,753	55,524	444.0
Connecticut	43,642	8,719	4,990	10.2	13,446	8,625	4.8	46,306	32,796	36.0	68,471	46,411	51.0
Delaware	24,456	11,309	8,607	23.8	3,624	2,315	19.2	13,997	7,953	64.7	28,930	18,875	107.7
Florida	20,116	57,509	44,321	268.4	10,776	6,827	54.1	75,028	54,367	333.9	143,313	105,515	656.4
Georgia	20,323	110,541	81,639	622.3	11,956	7,294	69.5	138,499	87,447	1,050.1	260,996	176,380	1,741.9
Idaho	38,174	16,873	14,159	97.1	2,874	2,436	33.6	28,267	20,585	222.9	48,014	37,180	353.6
Illinois	54,172	101,204	68,723	731.7	86,053	61,446	123.2	224,962	166,957	724.5	412,219	297,126	1,579.4
Indiana	78,480	71,060	53,191	243.8	56,045	33,868	359.8	61,605	40,203	249.1	188,710	127,262	852.7
Iowa	41,321	45,437	33,765	598.6	29,431	23,392	184.0	65,756	44,783	1,201.7	140,624	101,940	1,984.3
Kansas	39,363	26,804	20,691	814.7	24,240	16,687	303.3	65,478	45,363	1,326.6	116,522	82,741	2,444.6
Kentucky	51,582	48,830	35,057	127.6	2,717	1,861	11.6	76,183	52,935	286.1	127,730	89,853	425.3
Louisiana	26,862	53,861	35,769	336.0	51,577	37,250	207.3	68,028	41,475	360.3	173,466	114,494	903.6
Maine	26,775	11,034	8,649	12.8	4,280	3,847	2.6	26,485	17,892	103.3	41,799	30,388	118.7
Maryland	11,231	29,548	17,734	153.5	23,891	15,166	35.9	79,398	57,895	199.5	132,837	90,795	388.9
Massachusetts	32,610	46,479	35,991	21.2	44,136	24,215	27.7	106,280	74,489	62.7	196,895	134,695	111.6
Michigan	52,302	96,798	72,687	660.2	39,119	26,125	226.7	139,986	101,652	410.1	275,903	200,464	1,297.0
Minnesota	67,659	13,538	10,774	53.5	12,575	7,366	135.5	101,679	76,467	953.2	127,792	94,607	1,142.2
Mississippi	18,540	50,293	36,584	699.7	21,463	15,468	167.5	75,455	52,932	876.4	147,211	104,984	1,743.6
Missouri	41,708	42,005	27,215	1,339.1	21,242	16,266	46.7	122,613	83,090	1,075.0	185,860	126,571	2,460.8
Montana	68,913	12,166	8,314	171.2	4,231	3,565	24.3	49,531	36,721	439.3	65,928	48,600	634.8
Nebraska	63,825	10,511	6,684	229.0	11,304	5,967	154.7	64,512	42,337	1,152.6	86,327	54,988	1,536.3
Nevada	36,173	8,091	7,506	71.1	1,163	1,014	18.6	30,461	27,919	194.9	39,715	36,439	284.6
New Hampshire	21,370	4,612	2,529	16.8	1,918	1,405	3.4	25,979	17,832	71.0	32,509	21,766	91.2
New Jersey	88,281	18,620	10,152	64.1	19,977	15,162	10.6	83,060	58,711	48.0	121,657	84,025	122.7
New Mexico	13,309	10,264	9,115	37.0	6,389	5,576	65.7	49,860	40,298	339.8	66,513	54,989	442.5
New York	146,649	54,314	37,420	115.4	61,156	45,503	56.1	534,203	341,546	409.1	649,673	424,469	580.6
North Carolina	67,463	51,669	41,957	225.7	10,850	7,487	134.0	96,370	63,749	857.7	158,889	113,193	1,217.4
North Dakota	23,056	26,819	19,484	1,254.5	19,245	14,901	315.9	28,887	19,656	1,148.4	74,951	54,041	2,718.8
Ohio	39,434	48,674	26,514	118.4	41,561	31,755	73.1	278,927	200,621	355.1	369,162	258,890	546.6
Oklahoma	17,030	73,096	52,317	780.3	19,306	15,719	93.7	66,573	45,212	620.2	158,975	113,248	1,494.2
Oregon	34,360	7,904	5,422	76.3	12,145	10,093	67.9	45,039	35,766	248.5	65,088	51,281	392.7
Pennsylvania	105,908	109,426	75,110	216.2	77,406	52,039	95.0	263,564	177,142	369.7	450,396	304,291	680.9
Rhode Island	12,065	8,030	4,129	8.2	9,309	7,981	10.7	22,630	16,675	16.2	39,969	28,785	35.1
South Carolina	24,304	52,282	39,126	646.2	14,588	12,057	90.3	51,673	32,249	779.2	118,543	83,432	1,515.7
South Dakota	11,163	61,736	49,289	687.2	1,197	668	166.4	30,284	22,619	575.5	93,217	72,576	1,429.1
Tennessee	36,033	68,130	52,992	480.0	13,893	9,510	63.1	120,565	77,318	741.0	202,588	139,820	1,284.1
Texas	86,634	113,180	94,772	702.0	52,091	40,005	272.5	240,730	172,380	2,071.3	406,001	307,157	3,045.8
Utah	12,502	29,886	27,114	194.9	4,023	3,448	18.5	29,023	25,110	130.3	62,932	55,672	343.7
Vermont	7,820	20,702	18,171	27.5	401	200	.9	22,880	16,412	61.9	43,983	34,783	90.3
Virginia	21,469	61,722	48,678	195.0	2,224	1,115	11.0	105,182	77,165	324.8	169,128	126,958	530.8
Washington	33,194	26,420	18,761	218.9	13,553	9,343	44.2	72,985	55,544	272.2	112,958	83,648	535.3
West Virginia	40,738	50,091	36,764	89.4	5,340	2,745	15.0	40,876	23,130	74.8	96,307	62,639	179.2
Wisconsin	83,004	31,144	20,382	368.2	16,196	11,464	154.3	61,366	40,623	430.8	108,706	72,469	953.3
Wyoming	12,251	19,604	16,281	84.2	1,196	1,126	2.1	48,838	40,786	336.0	69,638	58,193	422.3
Hawaii	4,472	9,250	4,625	16.7	1,410	650	1.8	2,689	1,341	4.1	13,349	6,616	22.6
District of Columbia	17,665	17,760	14,138	9.2	2,380	1,247	2.1	28,783	21,809	5.0	48,923	37,194	16.3
Puerto Rico	10,308	16,087	7,785	25.2	2,006	986	3.8	19,310	8,914	58.1	37,403	17,685	87.1
Alaska	7,756	15,658	14,313	254.9	787	787	20.5	11,838	10,331	314.8	28,283	25,431	590.2
TOTAL	2,022,946	2,073,732	1,526,832	15,541.7	967,608	686,202	4,372.7	4,870,653	3,254,263	24,305.8	7,911,993	5,467,297	44,220.2

1/ Includes additional funds authorized by Federal-Aid Highway Act of 1958, apportioned April 16, 1958.

VOL. 30, NO. 3

AUGUST 1958

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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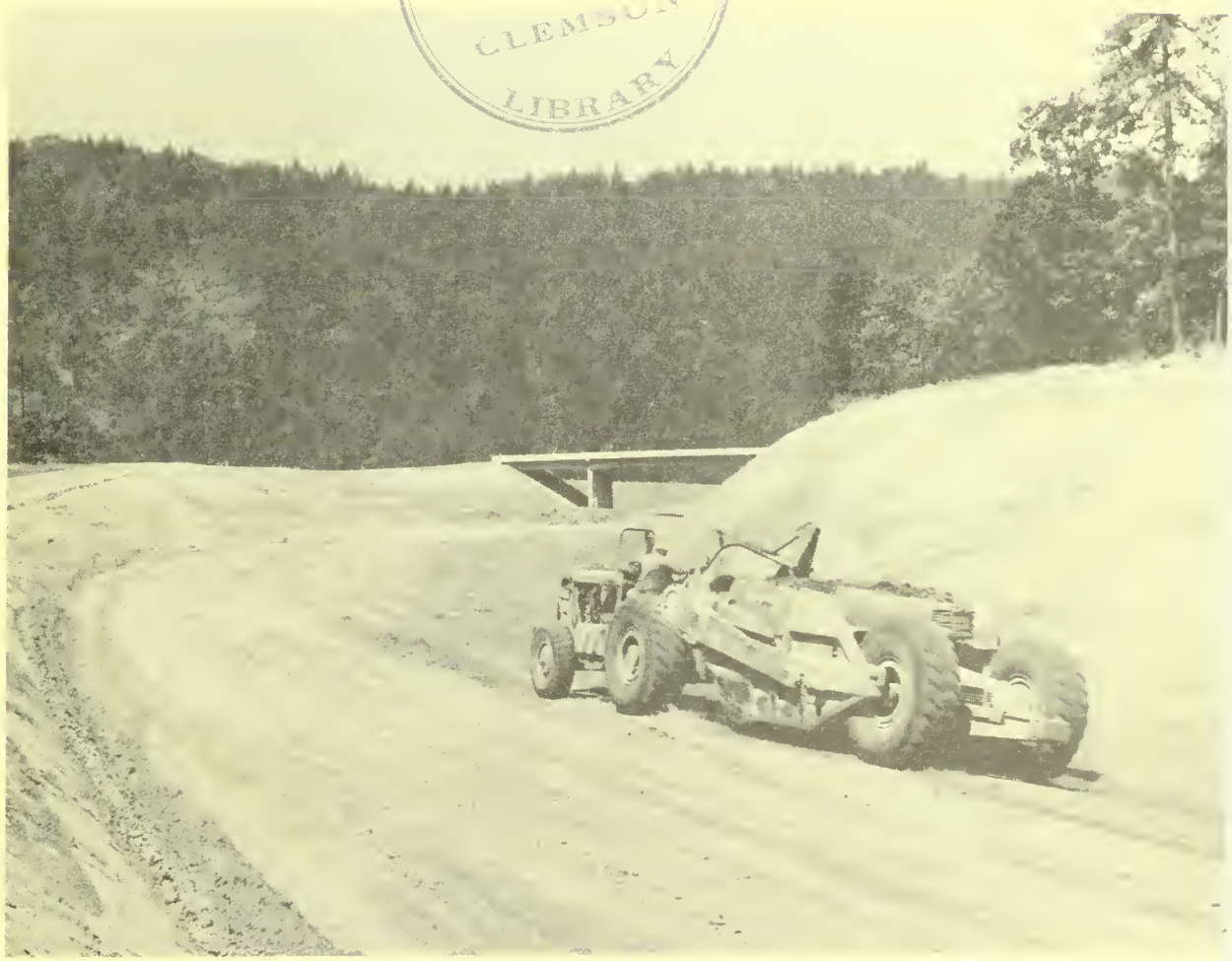

Grading of Interstate system highway, U. S. 99, between Roseburg and Myrtle Creek, Oreg.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30. No. 3 *August 1958*

C. M. Billingsley, *Editor*

BUREAU OF PUBLIC ROADS

Washington 25, D. C

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IN THIS ISSUE

Pore Pressures in Base Courses 53

A Rapid Method Utilizing Surface Area Measurements in Predicting the Amount of Cement Needed to Stabilize Plastic Soils 63

Lateral Placements of Trucks on Two-Lane Highways and Four-Lane Divided Highways. 71

New Publication: Highway Statistics, 1956 75

PUBLIC ROADS is sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1 per year (25 cents additional for foreign mailing) or 20 cents per single copy. Free distribution is limited to public officials actually engaged in planning or constructing highways, and to instructors of highway engineering. There are no vacancies in the free list at present.

Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

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Pore Pressures in Base Courses

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by EDWARD S. BARBER and GEORGE P. STEFFENS, Highway Physical Research Engineers

Although it is desirable to keep roadway base courses dry, they usually get wet in spite of efforts to prevent entrance of water or to provide drainage. However, the stability of granular material depends more on the state of stress in the pore water than on the amount of water present.

A load applied to wet soil (total pressure) is carried between the soil grains (intergranular pressure) or on the water (pore pressure). Since the strength of soil depends upon the intergranular pressure, any pore pressure reduces the strength because the intergranular pressure equals the total pressure minus the pore pressure.

Therefore, measures which prevent the development of pore pressures may maintain the stability of a base course even though the material stays wet. This requires minimizing the amount of fines without making a material too harsh to place with a uniform high density.

DURING the spring and summer on sunny days, water was often observed on the surface of an experimental flexible pavement (*1*, *2*)[2] built at Hybla Valley, Va. Beginning about 2 years after construction, water appeared during the early afternoon and disappeared in the evening. This water movement is not attributable to fluctuations of the water table in the subgrade because the test sections of pavement were built on a 5-foot clay fill, designed to provide uniform subgrade conditions during the period that static and moving load tests were scheduled. A similar phenomenon had been reported in Texas (*3*) and was attributed to pore pressures related to temperature changes in the base course. Related effects of pore pressures are given in published reports for bituminous surfaces (*4*), roofs (*5*), and paint (*6*).

To study this condition quantitatively, concurrent records were made of barometric pressures and temperatures and manometer pressures at various depths below the surface of the test pavement. Also fluctuations in the height of the water table in the base course were observed in holes dug through the pavement, and measurements of the height were made simultaneously with other readings for correlation purposes. Ample data on the moisture-density of the base course were available for correlation, because this study was conducted during the same period that load tests were made to evaluate the thicknesses of the various sections of flexible pavement.

This article presents the theoretical relation between temperature and pore pressure, its correlation with field measurements, and general observations of the effect of pore pressures on the stability of base courses.

Effects of Temperature Changes on Air-Water Mixtures

At temperatures above freezing, the voids in a base course are filled with air carrying various degrees of water vapor and water with dissolved air. The volume of the air-water components for any temperature and pressure condition can be calculated from the gas laws of Boyle, Charles, Dalton, and Henry. For example, assume a condition in a base course where the volume of voids for a given volume of base is 100 cc.; also assume that these voids are filled with 88 cc. of water saturated with dissolved air and 12 cc. of air saturated with water vapor at 32° F. and a pressure of one atmosphere (760 mm. of mercury).

From the properties of water taken from the International Critical Tables (*7*), shown in table 1, and molecular weights of 29 and 18 for air and water, respectively, the following calculations for several assumed temperatures and pressures indicate how the data used later in this article were developed for correlation purposes.

Using a specific volume of water 1.00013 (see table 1 for 32° F.), the weight of 88 cc. of water is 87.989 g. The vapor pressure at 32° F. is 4.58 mm., which leaves a partial pressure on the dry air of 755.42 mm. The weight of dissolved air by Henry's law, using the solubility of air in water (partial pressure per mole fraction) at 32° F. of 3.277 x 10^7, is 87.989 x

Table 1.—Physical properties of water

Temperature		Specific gravity	Specific volume	Viscosity	Surface tension	Saturated vapor		Air solubility, partial pressure per mole fraction
° F.	° C.					Pressure	Density	
				Dyne-sec./cm.² $\times 10^{-3}$	*Dynes/cm.*	*Mm. Hg.*	*Mg./cc.*	*Mm. Hg.* $\times 10^7$
32	0	0.99987	1.00013	17.94	75.64	4.579	0.00485	3.277
33.8	1	.99993	1.00007	17.32	75.50	4.926	.00519	3.361
35.6	2	.99997	1.00003	16.74	75.35	5.294	.00556	3.450
37.4	3	.99999	1.00001	16.19	75.21	5.685	.00595	3.536
39.2	4	1.00000	1.00000	15.68	75.07	6.101	.00636	3.624
41	5	.99999	1.00001	15.19	74.92	6.543	.00680	3.712
42.8	6	.99997	1.00003	14.73	74.78	7.013	.00726	3.803
44.6	7	.99993	1.00007	14.29	74.64	7.513	.00775	3.894
46.4	8	.99988	1.00012	13.87	74.50	8.045	.00827	3.985
48.2	9	.99981	1.00019	13.48	74.36	8.609	.00882	4.076
50	10	.99973	1.00027	13.10	74.22	9.209	.00941	4.168
51.8	11	.99963	1.00037	12.74	74.07	9.844	.01002	4.257
53.6	12	.99952	1.00048	12.39	73.93	10.518	.01067	4.347
55.4	13	.99940	1.00060	12.06	73.78	11.231	.01135	4.438
57.2	14	.99927	1.00073	11.75	73.64	11.987	.01206	4.525
59	15	.99913	1.00087	11.45	73.49	12.788	.01283	4.612
60.8	16	.99897	1.00103	11.16	73.34	13.634	.01364	4.701
62.6	17	.99880	1.00120	10.88	73.19	14.530	.01447	4.789
64.4	18	.99862	1.00138	10.60	73.05	15.477	.01536	4.874
66.2	19	.99843	1.00157	10.34	72.90	16.477	.01631	4.964
68	20	.99823	1.00177	10.09	72.75	17.535	.01730	5.044
69.8	21	.99802	1.00198	9.84	72.59	18.650	.01835	5.130
71.6	22	.99780	1.00221	9.61	72.44	19.827	.01942	5.216
73.4	23	.99757	1.00244	9.38	72.28	21.068	.02058	5.297
75.2	24	.99733	1.00268	9.16	72.13	22.377	.02178	5.379
77	25	.99707	1.00293	8.95	71.97	23.756	.02304	5.468
86	30	.99568	1.00434	8.00	71.18	31.824	.03035	5.858
95	35	.99406	1.00598	7.21	70.38	42.175	.03960	6.249
104	40	.99225	1.00782	6.54	69.56	55.324	.0511	6.611
113	45	.99024	1.00985	5.97	68.74	71.88	.0656	6.916
122	50	.98807	1.01207	5.49	67.91	92.51	.0832	7.188
131	55	.98573	1.01448	5.07	67.05	118.04	.1046	7.43
140	60	.98324	1.01705	4.70	66.18	149.38	.1305	7.64
149	65	.98059	1.01979	4.37	65.3	187.54	.1615	7.83
158	70	.97781	1.02270	4.07	64.4	233.7	.1984	7.98
167	75	.97489	1.02576	3.81	63.5	289.1	.2421	8.09
176	80	.97183	1.02899	3.57	62.6	355.1	.2938	8.17
185	85	.96865	1.03237	3.36	61.7	433.6	.3541	8.21
194	90	.96534	1.03590	3.17	60.8	525.8	.4241	8.22
203	95	.96192	1.03959	2.99	59.8	633.9	.505	8.20
212	100	.95838	1.04343	2.84	58.8	760.0	.598	8.16

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] Italic numbers in parentheses refer to references on page 62.

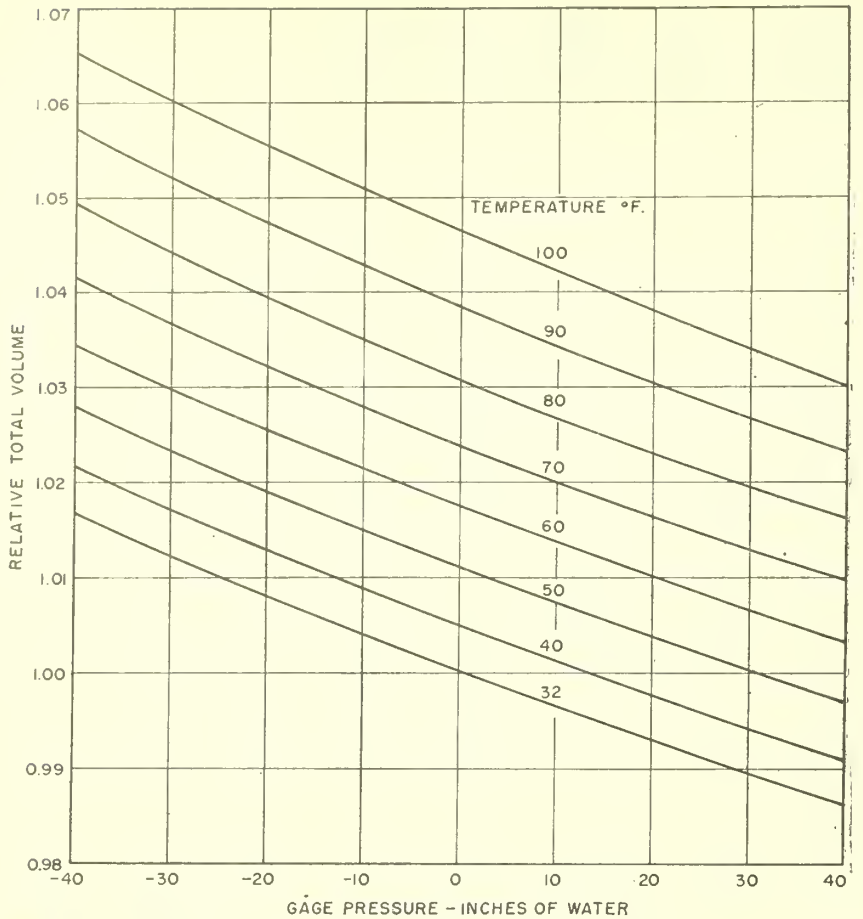


Figure 1.—Relation of volume and pressure for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

29/18 x 755.42/(3.277 x 10^7) = 0.00327 g. Since a molecular weight of air occupies 22,400 cc. at 760 mm., the density of dry air by Boyle's law is 29/22,400 x 755.42/760 = 0.001287 g./cc. Using this value, the weight of dry air in 12 cc. of saturated air is 0.001287 x 12 = 0.01544 g.

If the temperature is raised to 68° F. and the total pressure remains constant at 760 mm., the volume of water becomes 88.145 cc., the vapor pressure becomes 17.54 mm., and the partial pressure of the dry air is reduced to 742.46 mm. With the solubility coefficient decreased to 5.044 x 10^7, the weight of the dissolved air becomes 0.00209 g. and releases 0.00118 g. to make the total free air 0.01662 g. At 68° F. the density of dry air by Charles' law is 29/22,400 x 742.46/760 x 492/528 = 0.001180 g./cc. The volume of dry air, using this calculated value of density, is 0.01662/0.001180 = 14.080 cc. Thus the total volume becomes 102.23 cc. (88.145 + 14.080), which is an expansion of 2.23 percent in the air-water mixture when the temperature is changed from 32° F. to 68° F.

Using a coefficient of cubical expansion of stone 18 x 10^{-6} per degree F., this 36° F. temperature change would cause, at the most, an expansion of 0.07 percent (18 x 10^{-6} x 36) which is negligible compared to the 2.23-percent increase of volume in the air-water mixture from 32° F. to 68° F.

If the temperature is held constant at 68° F. while the pressure is increased 25.4 mm. (13.6 inches of water), the dissolved air in water is increased by 0.00209 x 25.4/742.46 = 0.00007 g. leaving 0.01655 g. of dry air. The volume of this weight of free dry air is 0.01655/0.001221 = 13.56 cc., and therefore the total volume of water and air becomes 101.71 cc.

Similar calculations for various temperatures and pressures provide data for figures 1, 2, and 3, which show graphically the relations between temperature, pressure, and volume for an air-water mixture of constant composition. In these calculations, zero gage pressure is taken as 760 mm. of mercury.

The rates of change of volume with temperature for various air-water compositions are shown in figure 4, and figure 5 shows the amount of free air at various temperatures for several air-water compositions. The relation between pressure increase and temperature increase for various compositions at a constant volume with zero initial gage pressure is shown in figure 6. In figure 4, it is noticed that the volume increase per degree F. increase becomes greater as the relative amount of free air increases. On the other hand, figure 6 shows that the pressure increase per degree F. increase is at a maximum for zero free air and decreases with greater amounts of free air.

Pore Pressures Related to Cyclical Temperature Changes

If the temperature varies with time, and conditions are such that there is no movement of water and escape of air from a base course, the changes in pore pressure will be proportional to the temperature change shown in figure 6 for various air-water mixtures. However, if the pressure changes in the voids and the conditions are such that the downward movement of air or water from a wide thin dense base course is obstructed, either by relatively impervious fine-grained subgrade material or a frozen saturated sand, it will slowly drain laterally with the tendency to move upward through or into the upper part of a less saturated base.

This latter condition simulates a system corresponding to that occurring in a one-dimensional consolidation test with one impervious face, i. e., impervious boundaries at the bottom and sides of the base course and a reservoir of high permeability at the top (surface), except that the coefficient of consolidation, c_v, is taken as the permeability divided by the compressibility of the air-water mixture in an incompressible base course, instead of the permeability divided by the compressibility of the base course with incompressible water. Let the base (thickness of H) be subjected to a sinusoidal temperature change of unit period with uniformity in temperature changes from top to bottom of the base, such that with no drainage the pressure would be $2\pi(t+x)$ as shown in figure 7.

Under these conditions, consider an increment of applied pressure, $dp = 2\pi \cos 2\pi(t+x)dt$ which is constant to time zero; at time zero, the effect of dp is reduced by the degree of consolidation to dp'. From equation 46, page 16 of Public Roads Magazine for March 1937, the reduced pressure is:

$$dp' = 2\pi \cos 2\pi(t+x) \sum_{n=1,3,5\ldots} \frac{4}{n\pi} e^{-\frac{n^2\pi^2 c_v t'}{4H^2}} \sin\frac{n\pi z}{2H}\, dt$$

where $t'(=-t)$ is the elapsed time from pressure application to time zero and z is the depth below the upper permeable boundary. Considering the impervious boundary ($z=H$) and expanding $\cos 2\pi(t+x)$ to ($\cos 2\pi t \cos 2\pi x - \sin 2\pi t \sin 2\pi x$), the total resulting pressure at time zero, assuming the application of pressure to have started at minus infinity, is:

$$p' = \sum_{m=1,-3,5\ldots} \frac{8}{m} \int_{-\infty}^{0} (\cos 2\pi t \cos 2\pi x - \sin 2\pi t \sin 2\pi x) e^{\frac{m^2\pi^2 c_v t}{4H^2}}\, dt.$$

By integration $p' = \sum_{m\ 1,-3,5\ldots} \frac{8}{m} \frac{\frac{m^2\pi^2}{4}\frac{c_v}{H^2}\cos 2\pi x + 2\pi \sin 2\pi x}{m^4 \frac{\pi^4}{16}\left(\frac{c_v}{H^2}\right)^2 + 4\pi^2}$

or $p' = A \cos 2\pi x + B \sin 2\pi x$

where A $\sum_{m=1,-3,5\ldots} \frac{\frac{m}{2}\left(\frac{c_v}{H^2}\right)}{1+\left(\frac{c_v}{H^2}\right)^2 \frac{\pi^2}{64} m^4}$

and B $\sum_{m=1,-3,5\ldots} \frac{4/\pi m}{1+\left(\frac{c_v}{H^2}\right)^2 \frac{\pi^2}{64} m^4}$

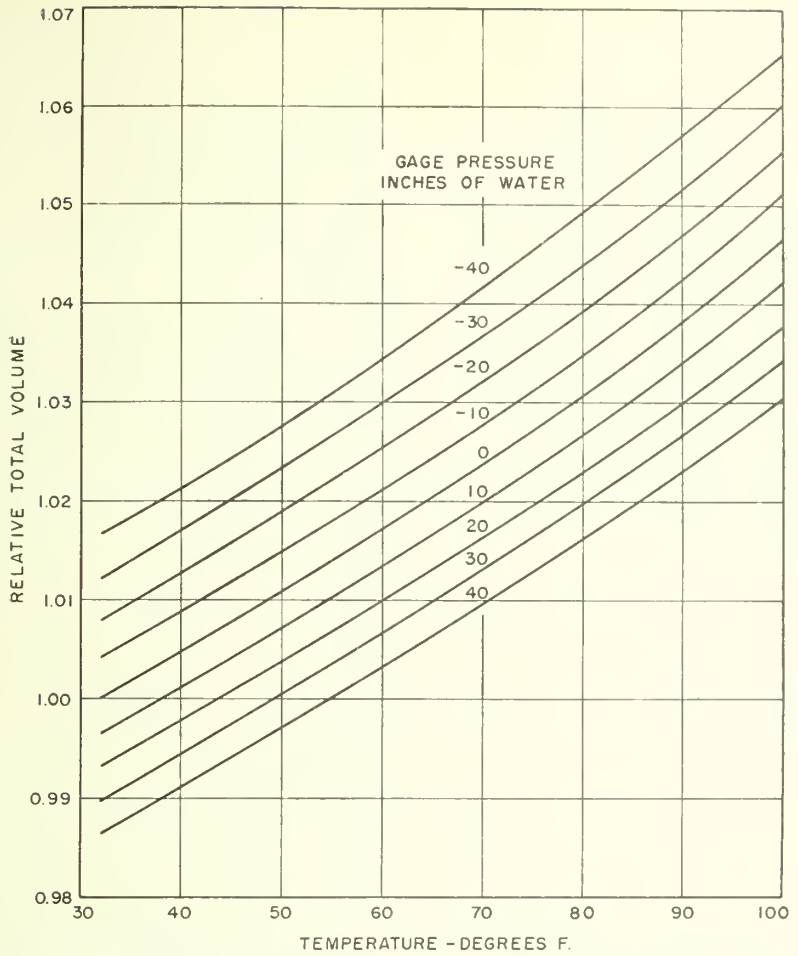


Figure 2.—Relation of volume and temperature for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

A and B have been numerically calculated for various values of c_v/H^2 as shown in table 2. By combination, $p'=\sqrt{A^2+B^2}\sin(2\pi x+\text{arc tan } A/B)$ which is a sine wave with an amplitude of $\sqrt{A^2+B^2}$ and a maximum occurring (arc tan $A/B)/2\pi$ before the maximum for no drainage. The calculated amplitudes ($\sqrt{A^2+B^2}$) and phase displacements (arc tan $A/B)/2\pi$ of the resultant sinusoidal pressure curves for various values of c_v/H^2 are shown in table 2.

In figure 8 the sinusoidal pressure curve for no drainage ($c_v/H^2=0$) is shown starting at time zero with a pressure of zero increasing to a maximum relative pressure of 1.00 at 0.25 cycle and then decreasing to zero pressure at 0.50 cycle. This curve is shown because it is used as a base for calculating the relative amplitudes and phase displacements for other values of c_v/H^2. The peaks of the curves for various drainabilities are shown in figure 8 by the points on the line denoted as locus of maximum pressures. For comparative purposes a complete half-cycle is also shown for $c_v/H^2=1$.

For zero permeability ($c_v/H^2=0$) there is, of course, no drainage of pore water and for very high permeability (high c_v/H^2) the pressure is lost by the escape of water. It is noted that when c_v/H^2 is less than 2, the maximum pressure is greater with drainage than with no drainage. Also the maximum pressure precedes the maximum temperature (pressure with no drainage) when c_v/H^2 is greater than 0.3.

Temperature and Pore Pressure Measurements

A flexible pavement, consisting of various thicknesses of gravel base (8 percent passing No. 200 sieve after construction) and a bituminous concrete surface, was constructed over a well-compacted fill of relatively impervious clay at Hybla Valley by the Bureau of Public Roads. A comprehensive testing program of static and moving loads was conducted over a period of about 5 years. Several years after the pavement was built, a series of pressure and water table fluctuation measurements were made in the base course because of the movement of free water from the base course through the bituminous concrete surfacing during the spring and summer.

Temperatures at various depths were measured with thermistors, and water pressures were measured with water manometers. Measurements of the fluctuations in the height of the free water table in the base course were recorded simultaneously with pressure and temperature readings to assemble data from which the graphs shown in figure 9 were developed (*8*). A study of the data shown in figure 9 indicates that the temperatures measured at the top and bottom of the base course are cyclic in nature, and that the amplitude of the temperature fluctuations decreases with depth below the top of the base course. The temperature at the bottom of the base course lags behind the top of the base because of the insulating effect of the base course.

The pore pressure, measured by the manometers, and the water level observed in the

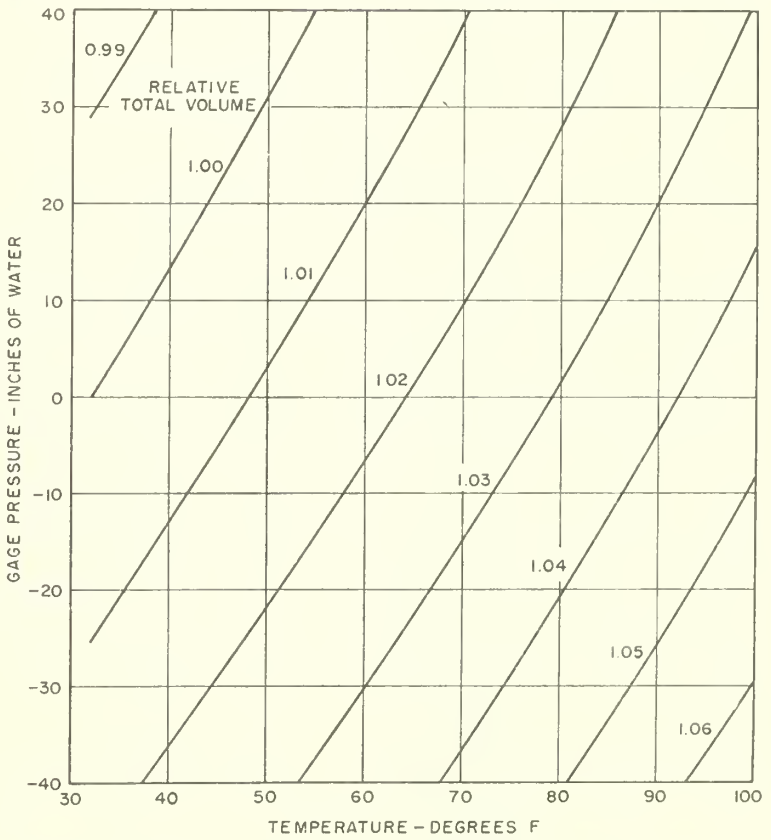


Figure 3.—Relation of pressure and temperature for 0.88 volume of water and 0.12 volume of air at 32° F. and zero gage pressure.

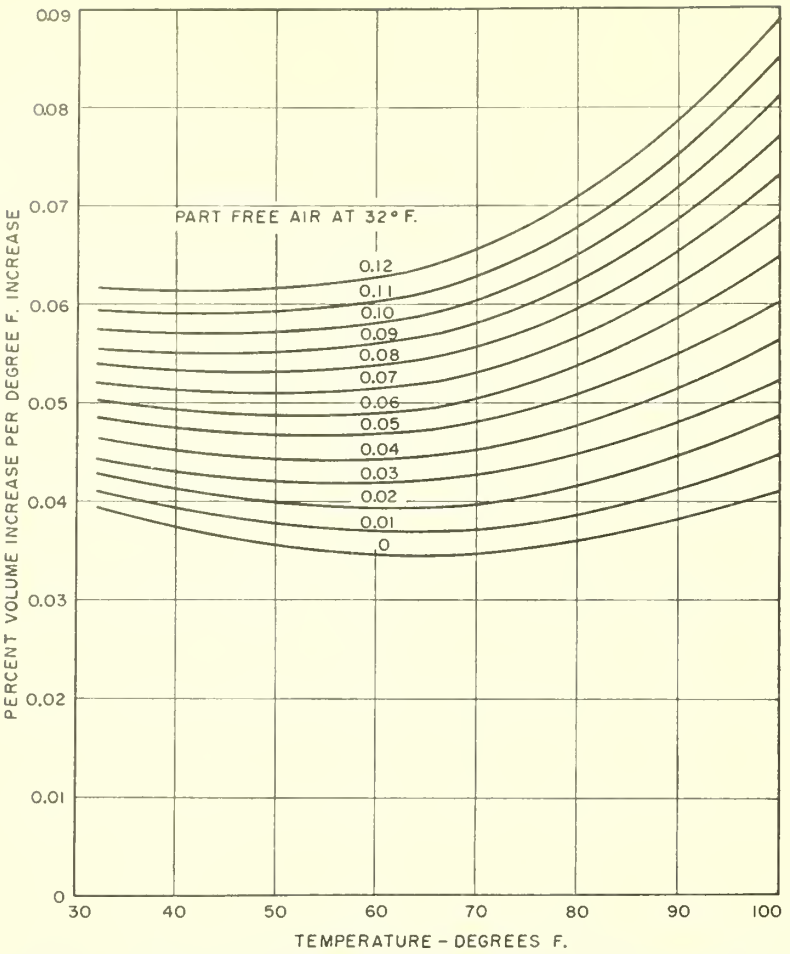


Figure 4.—Volume increase with temperature at zero gage pressure for various air-water mixtures.

4-inch diameter holes dug through the pavement and base course also show a cyclic fluctuation and follow the same general trend observed for the temperatures at the top and bottom of the base course. The height of water in the 4-inch diameter holes is a reflection of the water table but is considerably damped due to the capacity of the holes. The surface elevation of the pavement also increased slightly with increased temperatures.

The average of 16 measurements on the base course gave a dry density of 143.9 pounds per cubic foot and a moisture content of 5.3 percent. With a measured specific gravity of 2.66, the voids are computed to be 13.3 percent of the total volume. This average moisture content, calculated for 90° F. conditions, shows that the voids are comprised of 0.92 part of water. Using figure 5 this corresponds to about 0.06 part of free air at 32° F. and from figure 6, this corresponds to a pressure increase of 2.3 inches of water for one degree rise in temperature at approximately the 90° F. condition. The change in pore pressure for a no-drainage condition in the 12-inch base course can be calculated from the temperature and barometer readings shown in table 3.

For example, at 1 p. m. on August 26, 1954, the temperature increase from the initial readings (taken at 7 a. m. on August 25) is 3.7° F. (92.2–88.5); the corresponding pore pressure increase is 3.7 x 2.3=8.5. However, since the barometric pressure during this period changed 0.15 mm. (30.02–29.87), the pore pressure due to the temperature change of 3.7° F. is increased by 2.0 inches (0.15 x 13.6) giving a total of 10.5 inches which, added to the initial pressure reading of −0.3 inch, gives a calculated pore pressure of 10.2 inches The calculated pressures shown in the right-hand column of table 3 were computed in a similar manner and are compared with the observed manometer pressure readings in figure 10.

Water was observed at the surface of the 3-inch bituminous surface on the 12-inch base course on August 25. However, the maximum manometer reading observed was 13.3 inches during the same period. A study of the temperature and pressure data shown in figure 9 indicates that the temperature measured at the top of the base course increases more rapidly than the corresponding temperature measured at the bottom of the base course and is of a greater magnitude. This effect would tend to result in an increase in the pore pressure measured by the manometer at the bottom of the base course.

The lower values in the manometer readings can partially be explained by the fact that some flow of water is required before the manometer registers. Furthermore, as shown in figure 8, for example, at $c_v/H^2=1$, the maximum measured pressure develops before the maximum pressure calculated for a no-drainage condition in the base course ($c_v/H^2=0$). Although there is a time lag between the manometer readings and the calculated pore pressures for the no-drainage condition shown in figure 10, it is obvious that there is a direct relation between temperature and pore pressure occurring in the base course. Smaller pressure

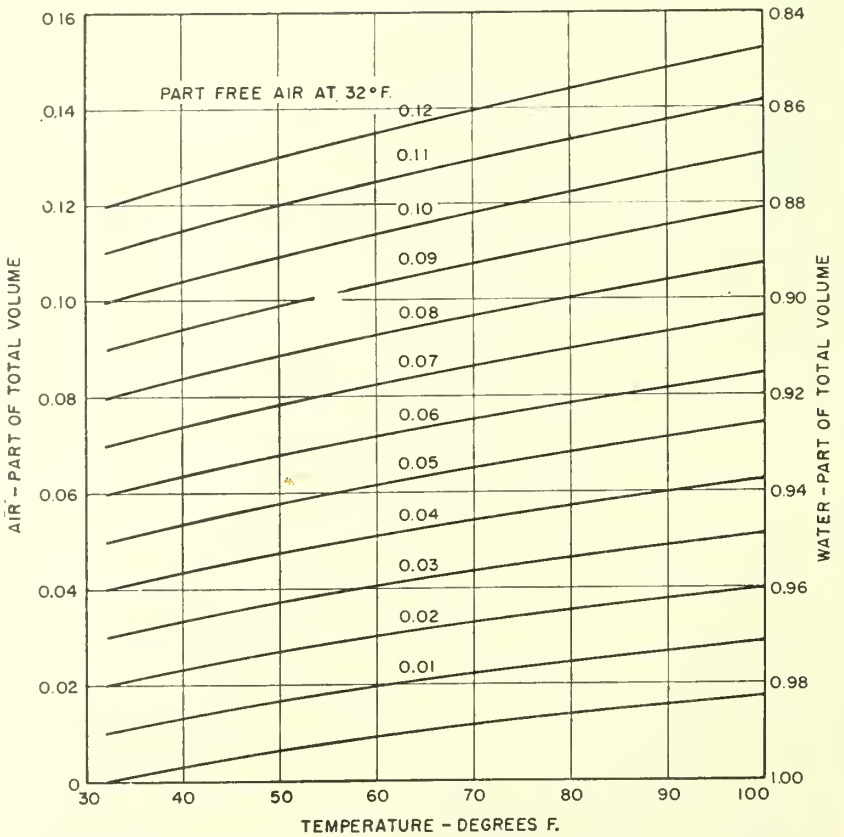


Figure 5.—Composition of air-water mixtures (zero gage pressure).

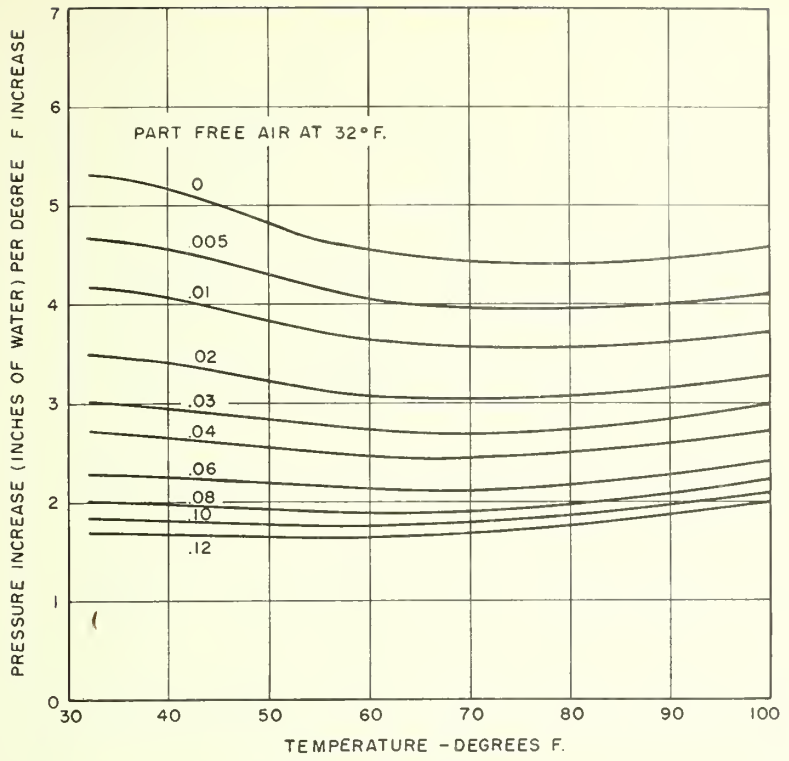


Figure 6.—Pressure increase with temperature at constant volume for various air-water mixtures.

and temperature changes were measured in an 18-inch base course, and much higher pressure changes would be expected in a 6-inch base course because of the greater temperature changes.

A series of observations of pressure and temperature were made over a period of 4 weeks, excluding weekends, and are shown in table 4 for comparative purposes. To evaluate these data, it was necessary to make several assumptions which are as follows:

(1) Constant climatic conditions were assumed prior to August 30 and linear interpolation was used to estimate conditions over weekends.

(2) The calculations of pressure were based, as before, on a constant air-water composition.

(3) Zero time lag was assumed in making the comparison of maximum measured pressures with maximum calculated pressures.

(4) A mean pressure of one atmosphere, 760 mm. of mercury, was assumed as a base for calculating daily cyclical changes in pore pressure.

(5) To compensate for the changing daily mean temperature and barometric pressure, a 4-day lag with a parabolic decrease with time was assumed to calculate the daily effective mean temperature and barometric pressure.

This adjustment is based on the fact that after the rains from September 17 to 21, the average manometer reading was above zero for about 4 days until the excess water which entered through the surface seeped out laterally. With the assumed 4-day lag and a parabolic decrease with time, the effective temperature and barometric differential for previous days is the sum of 0.55, 0.35, 0.20, and 0.10 times the respective change in the average of the barometer and temperature changes for the previous 1, 2, 3, and 4 days.

Hourly readings were taken from 8 a. m. to 11 p. m. on each weekday for the 4-week

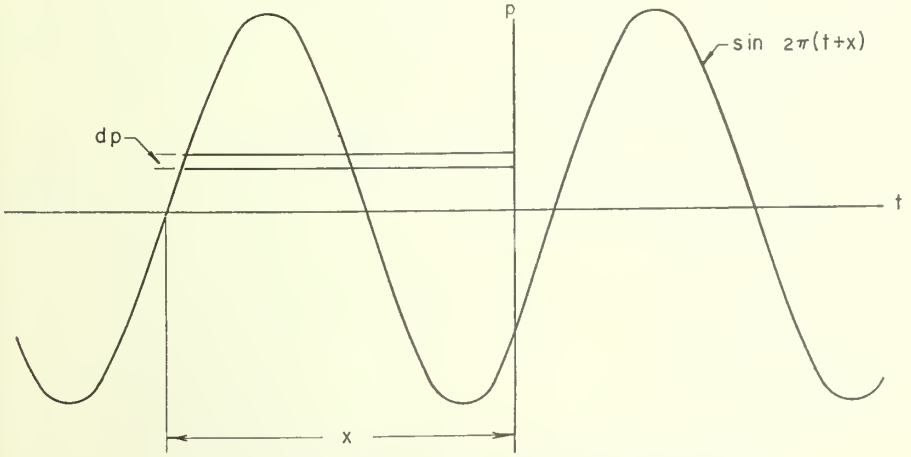


Figure 7.—Cyclical pressure applied with no drainage.

Table 2.—Factors for cyclical pressure change

c_v/H^2	A	B	$\sqrt{A^2+B^2}$	$\frac{\text{arc tan } A/B}{2\pi}$	Abscissa of maximum
0	0	1.000	1.000	0	0.250
0.1	−0.005	.994	.994	−0.001	.251
.2	−.028	1.026	1.026	−.004	.254
.3	−.007	1.078	1.078	−.001	.251
.5	.096	1.132	1.136	.013	.237
1	.341	1.074	1.127	.049	.201
2	.569	.780	.966	.100	.150
5	.495	.261	.559	.173	.077
10	.295	.077	.304	.209	.041
20	.154	.020	.156	.229	.021
50	.063	.003	.063	.242	.008
100	.031	.001	.031	.246	.004

period. The daily minimum and maximum temperatures at the bottom of the 12-inch base were obtained. Because of the limited amount of data, only about two-thirds of each daily cycle, it was necessary to use the ½ range of the daily temperature cycle (maximum−minimum)/2 and similarly the ½ range of daily barometric pressure cycle to develop data for correlation purposes.

The calculated ½ range in pressure is, as before, the sum of the barometer and temperature effect. It approximates the ½ range in manometer readings when they were determined (the manometer often failed to read the negative minimum pressure).

The temperature and barometer trends in terms of inches of water pressure are shown in table 4 and are reflected in the calculated effective temperature and barometer readings shown in figure 11. Adding the trend effects to the ½ range gives the calculated maximum pressures which are compared with the maximum manometer readings in figure 11. A good correlation is seen except for the period after rains.

The calculated values are for constant water content, so that any increase in water content due to infiltration of rainwater will temporarily make the measured pressures higher than those calculated. Thus, when the water depth in the 4-inch diameter hole rises (despite a fall in temperature) as on August 31 and September 20–22 due to the previous rain, the manometer readings are above the calculated pressures. This is especially noticeable for the last week where 3 or 4 days are required for the effects of infiltration to dissipate.

If the base course did not maintain almost a constant degree of saturation from top to bottom by capillarity, the volume change required to raise the water table would relieve much of the pressure. Figure 12 shows the moisture content after one day of drainage of a column of the passing No. 10 sieve fraction of the base course material, compacted to the equivalent density that the fraction passing the No. 10 sieve had in the compacted base course.

It is apparent from the data shown in figure 12 that the percentage of capillary water retained in the minus No. 10 sieve material in the base course is quite uniform (12.8 to 12.0 percent) for the 11-inch height above the free water table, measured for the 1-day drainage condition.

Effect of Pore Pressure on Theoretical Bearing Capacity

The effect of pore pressure on the theoretical static bearing capacity q of a base course may be calculated for various conditions from the formula and factors shown in figure 13 (*9*). A 9,000-pound wheel load on dual tires is assumed to produce a circular loaded area under the pavement surface with a radius a' of 5 inches and a pressure p' of 57 p. s. i. Assumed values for the components of the pavement are surface thickness $z=3$ inches, density of the base w' and surface $w=140$ pounds per cubic foot (0.081 lb. per cu. in.), and a coefficient of friction $f=1.0$ for a well-compacted base course.

Before applying the surface, z, a noncohesive base course material would have a bearing capacity of $q=0+0+0.081 \times 5 \times 192=78$ p. s. i. This provides very little margin of strength and makes such a material difficult to keep in place during construction. A damp stone or gravel base graded down to fine sand could develop a cohesion of 1 p. s. i. (2.3 feet of water) due to the surface tension of water. This would increase the bearing capacity to $q=1 \times 224+78=302$ p. s. i., which greatly increases its resistance to construction displacement. More cohesion could be obtained with the addition of a small amount of clay, but a slight excess would greatly reduce the friction and thereby decrease the bearing capacity when wet.

After surfacing, the moisture in the base course is apt to increase and thereby reduce the cohesion. With the water table at the bottom of a 12-inch base, the cohesion caused by surface tension would average 0.22 p. s. i. (6 inches of water). The bearing capacity would be $q=0.22 \times 224+0.081 \times 3 \times 172+78=169$ p. s. i., which is adequate to carry the imposed 9,000-pound wheel load. If a rise in temperature or leaks through the surface raised the water table to the top of the base, the cohesion would be lost; the effective density of the base w' would be reduced by the density of water and the bearing capacity would be reduced to $q=0+42+(0.081-0.036) \times 5 \times 192=85$ p. s. i., which leaves only a small margin of bearing capacity.

A temperature increase or other condition which raises the pore pressure to give an effective water table at the top of the pavement surface would further reduce the bearing capacity to $q=0+(0.081-0.036) \times 2 \times 172+(0.081-0.036) \times 5 \times 192=66$ p. s. i., which leaves very little margin of safety. Now suppose a temperature increase or an artesian condition raises the pore pressure to give an effective water table 6.7 inches above the top of the base. This reduces the surcharge effect of pavement to zero or, stated another

Table 3.—Calculation of pressure at bottom of 12-inch base course

Date, 1954	Hour, e. s. t.	Manometer reading	Temperature reading	Barometer reading	Calculated pressure[1]
		Inches of water	° *F.*	*Inches of mercury*	*Inches of water*
Aug. 25	7	−0.3	88.5	30.02	[2]−0.3
	9	−.1	87.7	30.02	−2.1
	11	2.3	87.4	30.00	−2.5
	13	8.9	88.8	29.97	1.1
	15	13.3	90.2	29.93	4.8
	17	13.1	92.0	29.89	9.5
	19	12.7	92.9	29.89	11.6
	21	9.3	93.8	29.91	13.4
	23	8.2	92.9	29.89	11.6
Aug. 26	1	6.1	92.2	29.87	10.2
	3	4.4	91.1	29.86	7.9
	5	2.9	90.8	29.87	7.0
	7	1.4	89.9	29.87	4.9
	9	.9	89.7	29.92	3.9
	11	2.4	89.4	29.92	3.2
	13	6.2	89.9	29.92	4.3
	15	12.7	91.1	29.90	7.3
	17	12.8	92.6	29.91	10.6
	19	11.5	93.5	29.92	12.6
	21	8.3	93.8	29.96	12.7
	23	5.2	93.2	29.99	10.9
Aug. 27	1	2.9	92.6	30.03	9.0
	3	1.1	91.4	30.03	6.3
	5	−.8	90.5	30.04	4.0
	7	−2.8	89.7	30.07	1.8
	9	−4.0	89.1	30.10	0
	11	−3.8	88.3	30.12	−2.2
	13	−3.4	88.5	30.13	−1.8

[1] Calculated on the basis of temperature and barometric pressure with constant air-water composition.
[2] Assumed to be equal to manometer reading.

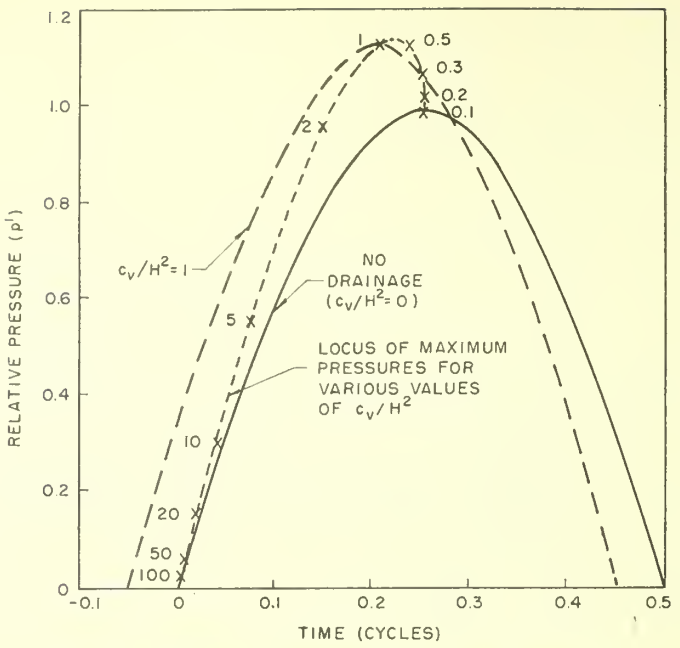


Figure 8.—Effect of drainage on cyclical pressure.

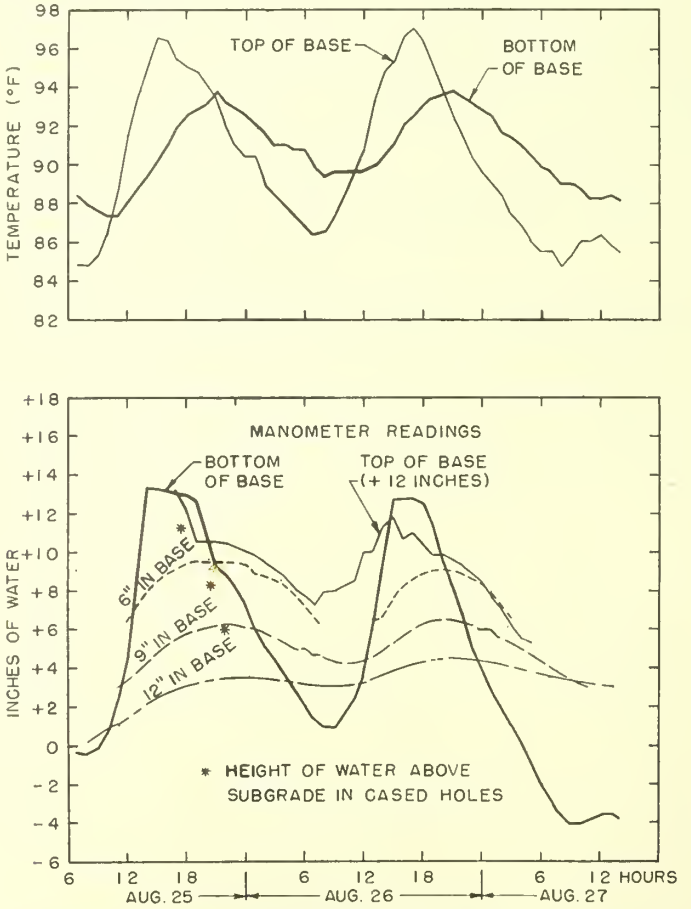


Figure 9.—Temperatures and water levels in 12-inch base course for a 55-hour period.

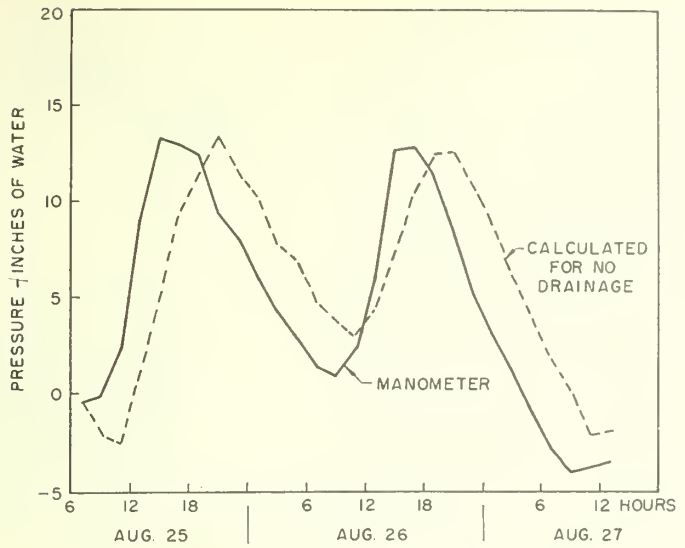


Figure 10.—Measured and calculated pore pressures.

way, the pavement is floating on a thin film of water between the base course and pavement. The bearing capacity of the base is thereby reduced to $q=0+0+(0.081-0.036)\times 5\times 192=43$ p. s. i., which indicates inadequate bearing capacity.

These calculations for static bearing capacity of a 12-inch granular noncohesive base course (see fig. 14) show reduction of bearing capacity from 169 p. s. i. to 66 p. s. i., when the effective water table rises from the bottom of the base course to the top of a 3-inch pavement surface. This change in effective water table (pore pressure) of 15 inches could be caused by an increase of the base course temperature of 6.5° F., as shown in previous calculations.

The bearing capacities just calculated may be compared with that of an open-graded base which stays sufficiently drained so that no cohesion or pore pressure is developed. This type of base (for $f=1$ and $c=0$) would have a constant bearing capacity of $q=0(224)+(0.081)(3)(172)+(0.081)(5)(192)=120$ p. s. i.

Laboratory Tests Showing Effect of Variables

The effect of pore pressure on the strength of granular materials can be demonstrated in the laboratory. For instance, CBR tests were run on compacted graded sand with respectively 100, 62, 39, 26, and 5 percent passing the number 4, 10, 20, 40, and 200 sieves. The results were as follows:

Water condition	*Surcharge, in pounds*	*Corrected CBR at 0.1-inch penetration*
Drained	10	63
Submerged	10	36
Submerged	5	18
Submerged	0	4

Figure 15 shows the decrease in penetration resistance of wet compacted sand when rapidly heated. Less effect is observed in cohesive soils; thus, a clay soil at 120° F. had two-thirds the penetration resistance it had at 40° F., which is less than the difference in the viscosity of water (*10*). At about 85-percent saturation the air permeability increases rapidly with less saturation, as shown in figure 16.

With increasing moisture content, the optimum moisture for dynamic compaction is reached when the escape of air and water is greatly restricted during the application of load. Therefore, the locus of optimum moistures for most soils and soil-aggregate mixtures, as shown in figure 17, is at approximately 85 percent of saturation. On the other hand, an optimum moisture is not obtained for materials with very high permeability or for compaction by long-time loading.

At a density of 94 pounds per cubic foot, the subgrade under the WASHO Road Test pavement (*11*) reached a critical condition when the moisture content rose to 23 percent. With a measured specific gravity of 2.55, this is a relative saturation of 85 percent.

Samples of this soil were tested in the soils laboratory of the Bureau of Public Roads. Figure 18 shows the relation between moisture and density for the WASHO subgrade soil and the penetration (0.001 inch of a 2-inch diameter piston loaded with 83 pounds and with 5,000-load applications) obtained for various percentages of saturation. A study of these data indicates that samples compacted to less than 85-percent saturation had lower values of penetration than those compacted to more than 85-percent saturation at equivalent moisture contents. Since penetration is a measure of strength, it is obvious that the bearing capacity of this soil is critical at or near 85 percent of saturation for dynamically compacted soils. However, it should be noted that soils statically compacted at the same moisture and density (see fig. 18) have much less penetration, and therefore have greater bearing capacity. The difference in these two methods of compaction is largely due to shearing during the dynamic compaction of this soil.

In addition to the adverse pore pressure developed with the increased percentage of saturation during loading, other related factors must be considered: (1) the effect of increased

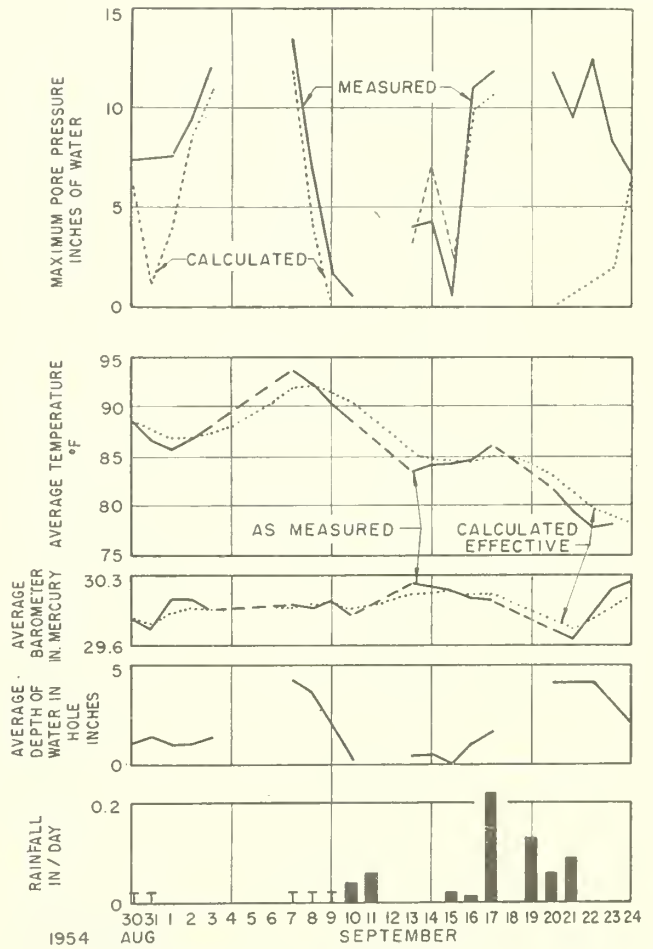
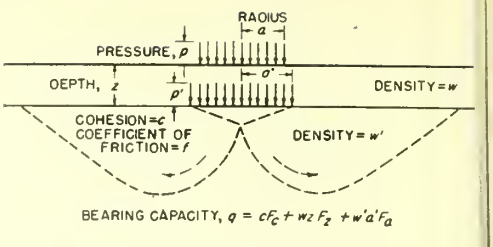


Figure 11.—Correlation of calculated pore pressure with measurements.

Table 4.—Pressures and temperatures for 4 weeks at bottom of 12-inch base course

Date	Barometer reading		Temperature reading		Calculated pressure, ½ range	Manometer reading		Temperature trend	Barometer trend	Calculated maximum pressure
	Mean, 8 to 18 hours	Difference at 18 hours	Mean, maximum and minimum	½ range x 2.3		½ range	Maximum			
	Inches of mercury	*Inches of water*	*°F.*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*	*Inches of water*
Aug. 1954:										
30	29.86	1.1	88.6	5.1	6.2	(1)	7.4	0	0	6.2
31	29.79	−1.0	86.7	4.3	3.3	5.2	7.5	−2.5	0.5	1.3
Sept. 1954:										
1	30.07	.4	85.6	8.6	9.0	(1)	7.6	−2.8	−1.8	4.4
2	30.08	1.0	86.7	9.4	10.4	(1)	9.5	−.4	−1.2	8.8
3	29.94	.5	88.0	8.5	9.0	(1)	12.1	1.6	.3	10.9
7	29.98	.7	93.5	7.5	8.2	8.6	13.5	3.8	−.2	11.8
8	30.01	.1	92.3	4.1	4.2	5.7	6.9	.6	−.3	4.5
9	30.06	.4	90.3	2.6	3.0	4.0	1.8	−2.6	−.6	−.2
10	29.90	1.1	88.4	2.3	3.4	(1)	.6	−4.2	.9	.1
13	30.24	1.2	83.3	8.4	9.6	(1)	4.1	−4.8	−1.5	3.3
14	30.14	.5	84.1	8.3	8.8	(1)	4.3	−1.5	−.3	7.0
15	30.17	.5	84.2	2.5	3.0	(1)	.7	−.4	−.2	2.4
16	30.08	.8	84.5	8.1	8.9	6.3	11.1	.5	.6	10.0
17	30.06	.7	86.0	7.1	7.8	8.3	11.9	2.4	.6	10.8
20	29.73	−.1	81.7	1.6	1.5	5.2	11.8	−3.3	1.7	−.1
21	29.63	1.4	79.1	2.8	4.2	5.0	9.6	−5.4	1.7	.5
22	29.93	−.3	77.8	7.4	7.1	7.7	12.5	−4.7	−1.3	1.1
23	30.19	.3	77.9	7.1	7.4	7.3	8.3	−2.4	−2.9	2.1
24	30.26	1.1	77.7	9.8	10.9	(1)	6.5	−1.4	−2.4	7.1

[1] Manometer failed to read minimum pressure.



f	Fc	Fz	Fa
0	7.4	0.0	0.0
0.1	9	0.8	0.1
0.2	14	2.1	0.4
0.3	19	4.3	1.2
0.4	27	8	3.0
0.5	36	14	6.6
0.6	53	24	13
0.7	75	40	26
0.8	106	67	51
0.9	156	108	102
1.0	224	172	192

Figure 13.—Bearing capacity under a circular loaded area.

percentages of granular material, (2) the increased coefficient of friction resulting from increase in angularity of the granular components of the soil, (3) the effect of density, and (4) the effect of varying the amounts of soil fines (passing No. 200 sieve) in granular materials.

Triaxial strength test data reported for pervious granular materials (*12*) show that the total strength of these materials increases up to 20 percent when the percentage of coarse aggregate above the No. 4 sieve is increased from zero to 50 percent. Also when the angularity of the coarse material was changed from rounded to crushed particles, there was an additional 10-percent increase in the coefficient of friction. However, the largest variable affecting the strength obtained was the density at which the specimens were tested. The average coefficients of friction obtained by the triaxial compression test for materials of various gradations were 0.76, 0.85, and 0.98 for relative densities of 0.50, 0.70, and 0.90, respectively.

Another investigator (*13*) reported similar advantage of densification of well-graded materials containing 10 percent passing the No. 200 sieve. With 6-percent water and lateral pressures up to 30 p. s. i., the triaxial tests indicated relative strengths of 0.58, 0.85, and 1.00 for dry densities of 126, 136, and 141 pounds per cubic foot, respectively.

The following data taken from an unpublished portion of a report (*14*) show the effect of decreasing the percentage of material passing the No. 200 sieve on compressive strength (lateral pressure equals 20 p. s. i.) and stress-strain modulus, as measured by the triaxial compressive strength test:

Percentage passing No. 200 sieve	*Compressive strength, p. s. i.*	*Stress-strain modulus, kips/sq. in.*
15	190	5
8	210	8
0	210	11

A study of these data indicates that there is a considerable increase in the stress-strain

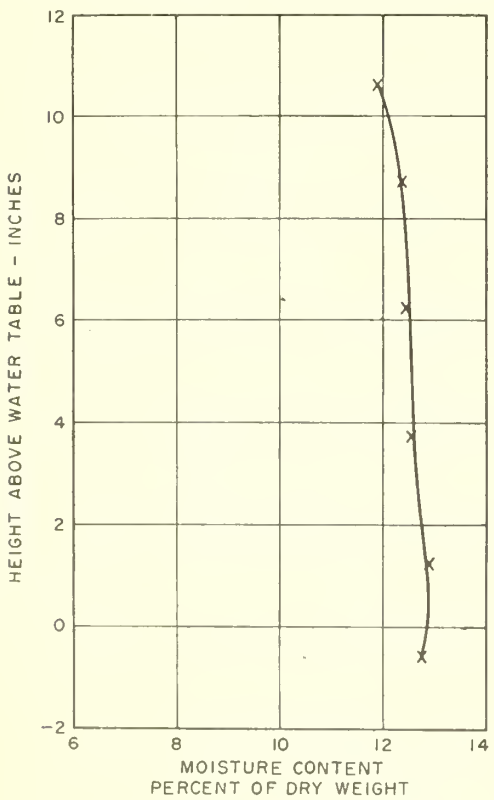


Figure 12.—Capillary water retention 24 hours after compaction.

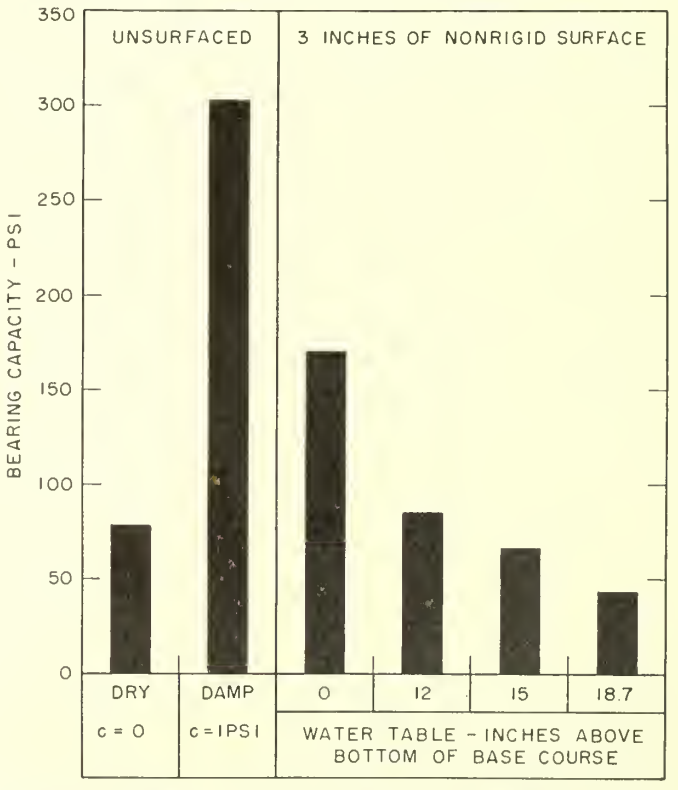


Figure 14.—Effect of pore pressure on calculated bearing capacity of a 12-inch base course (w=w'=140 lb./cu. ft., f=1.0, a'=5 inches).

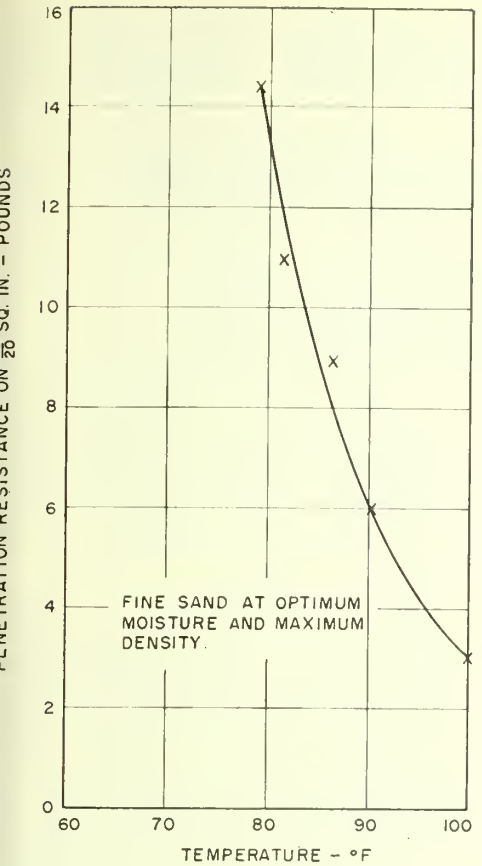


Figure 15.—Temperature related to penetration resistance.

modulus with a decrease in the percentage of soil passing the No. 200 sieve.

A third investigator (*15*) reported that a maximum density of a soil-gravel mixture was obtained with 10 percent passing the No. 200 sieve and a maximum CBR when 7 percent was used.

Field Conditions Where Pore Pressure Is a Factor

Critical pore pressures which affect the bearing capacity of a subgrade or base course can be controlled by changing the gradation and drainage so that the material, when compacted, has void spaces which are less than 80-percent saturated.

During construction, developed pore pressures may cause shearing which reduces the strength of soils in spite of the precautions taken during normal compaction. This effect may be minimized by compacting the soil at moisture contents slightly below standard AASHO optimum for low volume change soils, and by using larger roller contact areas to lower unit pressures. For earth dams, compaction 2 percent less than optimum is specified to minimize critical pore pressures in the structure. However, too low an initial moisture content in clay soils will usually cause swelling when the water content increases.

A well-drained condition requires a means of egress of water greater than the potential infiltration from the edges and through the surface. Excess water in a base may be removed downward by flow into a permeable subgrade with a deep water table or by lateral flow through a thick permeable base with sufficient outlets to reduce the required distance of lateral flow. Dense-graded bases of usual dimensions generally have insufficient permeability for rapid lateral drainage if saturated.

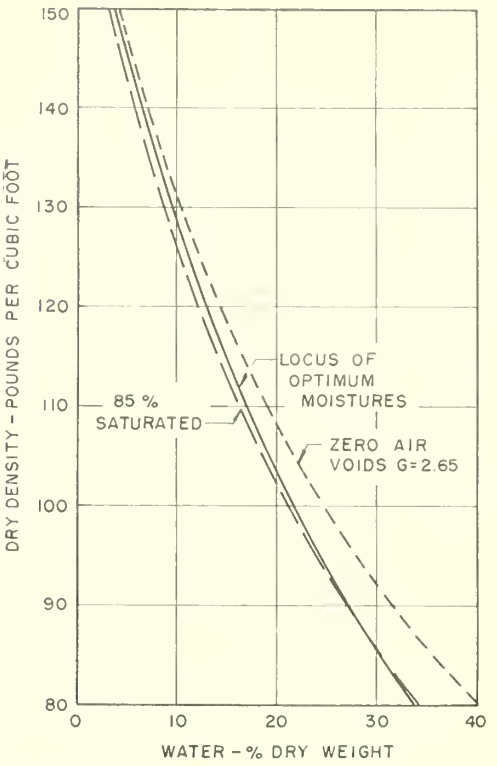


Figure 17.—Optimum moisture related to saturation.

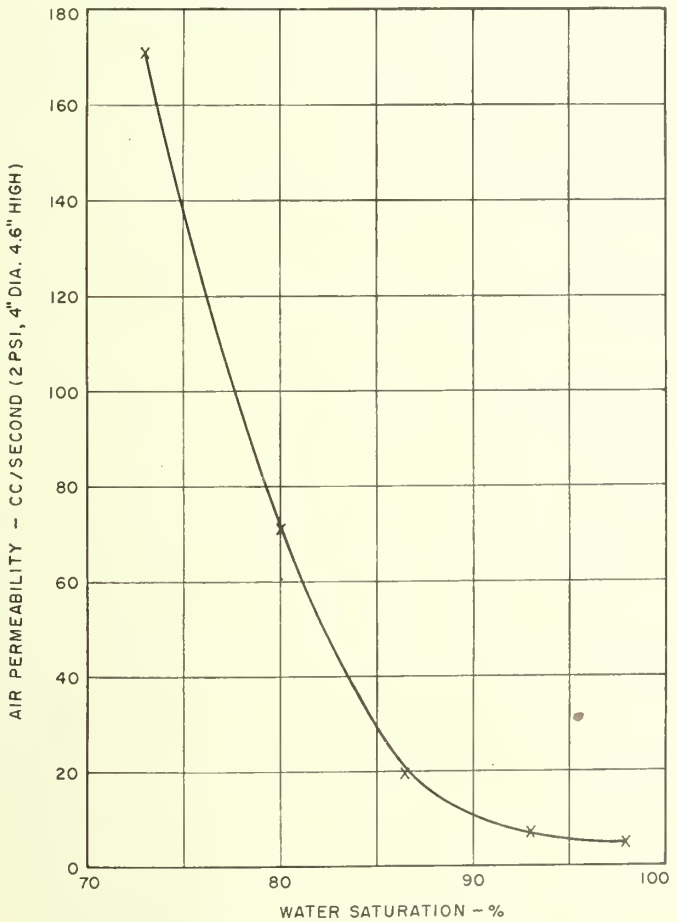


Figure 16.—Air permeability related to saturation.

In nonfrost areas, dense-graded materials have *sometimes* stood up even with poor drainage. This may be attributable to the tendency for a dense granular material to expand during shearing to overcome the interlocking obtained with compaction beyond the critical density. Thus, a 10° F. rise in temperature will cause a pressure of 23 inches of water, assuming 6 percent air at 32° F., but if the material expands 0.6 percent of its volume, the pressure is relieved.

This volume change is within the range of expansion of graded granular material compacted to 100 percent of standard compaction. Under dynamic loads, the effect of a standing water table may possibly be counteracted by this same expansion, if loads are not repeated at the same place, since high density with limited permeability causes a temporary vacuum to be created by shear expansion.

In regions where deep frost penetration occurs, thawing from the surface will usually prevent downward drainage of the base course for a period of time. Freezing and thawing of a very wet base course may adversely affect its stability, unless it has sufficient permeability when it thaws to allow water to escape from under a wheel load without lateral flow of the granular base. This requirement is demonstrated by materials which do not shove in the standard impact compaction test at moisture contents greater than the optimum moisture content (*16*). This criterion gener-

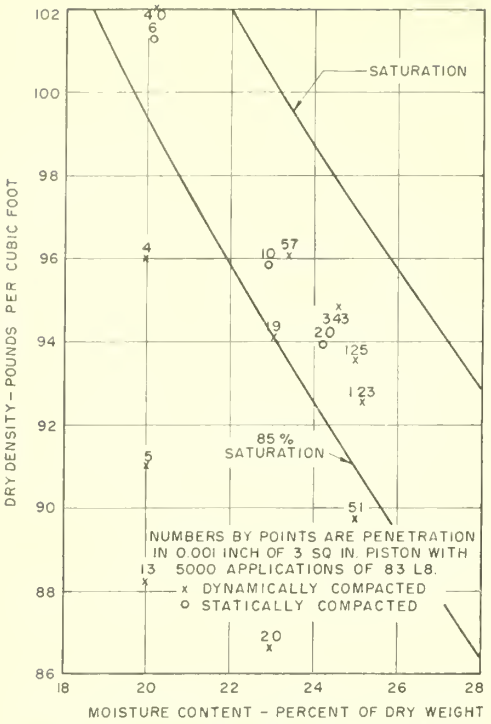

DRY DENSITY – POUNDS PER CUBIC FOOT
MOISTURE CONTENT – PERCENT OF DRY WEIGHT
SATURATION
85 % SATURATION
NUMBERS BY POINTS ARE PENETRATION IN 0.001 INCH OF 3 SQ IN. PISTON WITH 5000 APPLICATIONS OF 83 LB.
x DYNAMICALLY COMPACTED
o STATICALLY COMPACTED

Figure 18.—Penetration related to moisture and density.

ally limits the amount of fines to a maximum of 10 percent passing the No. 200 sieve, and this limit is used in several States. In North Carolina, extra-thick surfaces are required for bases with more than 10 percent passing the No. 200 sieve (*17*). Degradation during and after construction must be considered.

If open-graded base courses are used as insulation against frost, care must be taken to prevent ventilation through the base which can nullify the insulating effect. Such ventilation is the apparent reason for increased frost penetration into the subgrade with increased pavement thickness at the WASHO Road Test Project (*11*). The density of 132 pounds per cubic foot with an average moisture content of 5 percent corresponds to a degree of saturation of 50 percent which indicates a high air permeability.

Further research is required to develop more refined design criteria for base courses. This should include the thickness, width, and slope of base courses, the permeability, capillarity, strength and stress-strain relations of base course materials, as well as the ambient conditions of moisture and temperature.

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A Rapid Method Utilizing Surface Area Measurements in Predicting the Amount of Cement Needed to Stabilize Plastic Soils

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The relation between the cement requirement of soil-cement mixtures and surface area determined by the glycerol retention method was investigated for a group of plastic soils. For soils containing less than 45 percent silt, a strong correlation was found between surface area and the cement content at which a 10-percent loss occurred during 12 cycles of the freeze-thaw durability test.

A regression equation and procedure were developed for predicting the cement requirement from surface area. In a comparison of predicted cement requirements with those derived from freeze-thaw test data, an average deviation of 0.6 percent cement was obtained.

THE USE of soil cement for highway base courses has increased rapidly in recent years, especially in areas where local sources of granular deposits are nonexistent or rapidly being depleted. Present methods for determining the amount of cement necessary to produce a stable material from a given soil involve the use of rather large soil samples and an extended testing program. The freeze-thaw or wet-dry durability tests (*1*)[2] in particular require considerable time and effort. A short-cut method developed by the Portland Cement Association (P. C. A.) provides an easier alternate procedure for sandy soils, but extended testing is still required for plastic materials. The present article is concerned primarily with these plastic materials, and with the development of a shorter, more economical test procedure for estimating their cement requirements.

Despite a number of years of successful use of soil cement, little is known of the physical chemistry of the reactions between the soil and the cement. It has been noted that the more plastic soils generally require larger quantities of cement for effective stabilization. Although there does not seem to be any consistent relation between cement requirement and plasticity index or, indeed, any of the familiar engineering tests, it was thought that perhaps such a relation might exist with the surface area of the soil.

Some years ago, Catton (*2*), exploring this proposed relation by the use of surface area values computed from grain-size distribution curves, found an almost complete lack of correlation. However, an examination of Catton's data reveals that the largest values he computed for surface area were just over 2 square meters per gram (m.²/g.), even for A–6 and A–7 soils. Present knowledge indicates that actual surface areas are many times higher than this, often as high as several hundred m.²/g. Catton's computed values of surface area could not therefore be expected to provide any real correlation.

The method widely considered as standard for realistically determining the surface area of fine-grained materials is known as the Brunauer-Emmett-Teller (B. E. T.) Method, and involves the adsorption of nitrogen or some other gas on the surface of the material at low temperatures. The complicated apparatus and time-consuming procedures required, however, are not suitable for general routine determinations. Further, the method is not particularly applicable to soils since the nonpolar gases normally used do not measure the internal surfaces of expanding clay minerals which are present in many soils. The crystals of the clay minerals are formed in sheetlike layers; in an expanding mineral the flat surfaces of the inner layers constitute the internal surface.

Lately, a method (*3*) involving the retention of ethylene glycol by soils has also been used to estimate surface area, but there is some doubt as to the specific quantitative relation between the actual surface area and the amount of ethylene glycol retained under the conditions of the determination. More recently, a new and simpler method (*4, 5*) involving the retention of glycerol has been developed at the laboratories of the Bureau of Public Roads.

Values of surface area obtained for various clays by this method agree closely with theoretical values of expanding clays and, for nonexpanding clays, with those determined by the B. E. T. Method. In the present work, surface areas were measured by this method on a number of soils of known cement requirement. The correlation between the resulting surface area values and cement requirements was then determined. An equation was derived for predicting cement requirements from surface area values, and the predicted cement requirements were compared with the actual cement requirements as determined by test.

Conclusions

The following conclusions are drawn from this investigation:

1. For soils of measurable plasticity, a definite correlation exists between surface area, as measured by the glycerol retention procedure, and cement requirement (percent by volume) calculated from loss data of the freeze-thaw durability test and amended by other engineering considerations. However, this correlation is not sufficiently close to permit adequate predictions to be made directly from the surface area values. The weakness of this correlation is considered to be due to such factors as:

(*a*) The surface area expressed on a weight basis in contrast to the volume basis of the cement requirement.

(*b*) The different standards of allowable losses in the freeze-thaw test, 7, 10, or 14 percent, depending on the AASHO classification of the sample.

(*c*) The use of a certain amount of engineering judgment in deriving practical recommendations for field use from the freeze-thaw data.

(*d*) The rounding of the cement requirement to the nearest one-half percent.

(*e*) The presence of several soils high in silt content in the group of samples studied. Such soils have been shown to have cement requirements which do not correlate with surface area.

2. A very strong correlation is obtained when soils of higher than 45 percent silt are

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] Italic numbers in parentheses refer to the list of references on pages 69–70.

excluded from consideration, and the cement factor in the correlation is taken as the actual cement content by weight at which a 10-percent loss occurs in the freeze-thaw test, no allowance being made for AASHO class differences. The regression equation $y=0.087$ (surface area)$+3.79$ can be employed to derive accurate predictions of this cement factor from measurements of surface area by the glycerol retention method.

3. A suitable prediction of the conventional cement requirement, in percent by volume, can then be made by the following procedure: (*a*) modifying, if necessary, the cement factor predicted by the regression equation by adding 2.0 percent cement to adjust to the basis of a 7-percent allowable loss in the freeze-thaw test, or by subtracting 0.7 percent to adjust to the basis of an allowable loss of 14 percent; and (*b*) converting the modified value to a percent by volume basis using the density of the soil-cement mixture.

In a comparison of cement requirement values obtained by test with values obtained by the use of this procedure, the average deviation for the group of samples used in this study was 0.6 percent cement by volume, and considerably less than this for the more highly plastic soils of the group.

4. Admittedly, these results were obtained from a restricted number of samples and the procedure should be checked further with a wider and more representative selection of soils. Furthermore, use of the surface area determination to predict cement requirements should be accompanied by compressive strength or other tests on small specimens made at and near the predicted cement requirement.

Materials and Methods

Soils

Of the soils furnished for study,[3] only those with measurable plasticity indexes were included in this analysis. The soils of the first group received (group A) are described in table 1. They were not selected as representative of any particular soil area or type of soil, but were simply those on hand at the P. C. A. laboratory at the time this study was initiated. They do, however, include samples from a number of different States and of various soil horizons. The last 10 soils listed in table 1 (group B) were received as a second group and represent samples processed by the P. C. A. laboratory during the period in which this study was being made.

[3] Soil samples and accompanying test data were supplied by the Portland Cement Association. Messrs. J. A. Leadabrand and L. T. Norling of the Soil Cement Bureau were particularly helpful to the authors in conducting the present study.

Surface area measurements

In the glycerol retention method, advantage is taken of the ability of clay and other soil constituents to adsorb glycerol molecules on their surfaces. Conditions are maintained under which only a single layer of the glycerol molecules is adsorbed and retained. The amount of glycerol adsorbed is measured by weighing the sample before and after treatment, and the weight of the adsorbed glycerol can be related to the surface area of the sample.

As applied to this study, the method involves the following steps:

(1) Dry duplicate small samples (about 1 gram each) of the passing 40-mesh fraction of the soil at 110° C. in aluminum foil dishes, and

Table 1.—Description of soils

Identification						Physical properties [1]												Surface area of whole soil
						Gradation: percentage finer than—												
B.P.R. soil No.	P.C.A. soil No.	State	County	Soil series	Soil horizon	4-mesh sieve	10-mesh sieve	40-mesh sieve	200-mesh sieve	0.05 mm.	0.005 mm.	0.002 mm.	Liquid limit	Plasticity index	AASHO classification	Optimum moisture of soil-cement mixture	Maximum density of soil-cement mixture	Cement requirement, P.C.A.	
																Percent	*Lb./cu. ft.*	*Percent by volume*	*m.²/g.*
Group A Soils																			
S-32044	7538	Alabama	Madison		-------	100	85	61	24	21	6	5	17	4	A-2-4 (0)	11.4	115.5	7.0	12.5
S-32045	7494	do	Montgomery		B	100	100	98	70	48	29	28	35	11	A-6 (7)	17.7	104.0	13.5	90
S-32046	7537	do	do		B	100	100	94	43	38	29	27	34	12	A-6 (2)	17.9	108.0	10.5	49.5
S-32047	7509	Arkansas	Ashley	Richland	A, B, C	100	100	98	96	92	27	20	32	11	A-6 (9)	16.2	105.5	9.5	63
S-32050	7489	do	do	Portland	A	100	99	95	92	90	23	15	27	6	A-4 (8)	17.3	103.5	17±	59
S-32051	7490	do	do	do	B	100	100	97	94	90	23	18	30	9	A-4 (8)	18.2	104.7	11.0	79
S-32053	7552	do	Nevada	Ruston	C	100	100	70	30	28	22	18	39	17	A-2-6 (1)	15.0	109.0	7.5	48
S-32055	7555	do	do	do	A, B, C	100	99	99	38	32	22	18	20	6	A-2-4 (0)	12.7	119.0	7.0	37.5
S-32056	7542	do	White		A	100	99	97	76	56	19	12	22	5	A-4 (8)	12.3	112.6	11.0	36
S-32060	7375	Idaho	Idaho		-------	55	27	15	9	9	2	2	26	5	A-1-a (0)	10.0	134.7	9.0	14
S-32061	7497	Illinois	Cook		-------	55	47	35	19	15	7	5	25	6	A-1-b (0)	8.0	125.3	7.0	12
S-32062	7498	do	do		-------	100	99	97	91	88	42	28	41	16	A-7-6 (11)	23.4	96.7	14.5	81
S-32063	7-h	do	do		-------	100	100	97	90	87	48	--	41	24	A-7-6 (14)	17.5	104.5	13.0	77.5
S-32064	7460	do	Henry		-------	59	46	28	16	16	8	6	26	6	A-1-b (0)	9.4	130.3	7.0	7.5
S-32065	7558	do	Iroquois	Hagener	-------	85	83	78	33	22	9	3	18	7	A-2-4 (0)	10.2	121.0	7.5	25.5
S-32066	7528	do	Massac		-------	80	74	67	56	49	10	7	22	1	A-4 (4)	11.3	117.6	8.5	26
S-32067	7529	do	do		-------	77	71	66	60	58	16	13	26	6	A-4 (5)	13.3	115.8	10.5	40.5
S-32068	7530	do	do		-------	75	63	50	35	32	9	8	19	4	A-2-4 (0)	11.0	120.8	9.0	28.5
S-32069	7560	do	do		-------	55	48	41	36	34	11	8	30	9	A-4 (0)	11.2	119.5	7.0	22.5
S-32070	7561	do	do		-------	84	76	73	39	23	8	8	20	3	A-4 (1)	11.7	118.6	7.0	19.5
S-32071	7562	do	do		-------	80	71	66	56	51	15	12	26	5	A-4 (4)	13.0	117.4	9.5	37.5
S-32072	7563	do	do		-------	55	47	34	12	11	8	8	28	15	A-2-6 (0)	9.4	126.8	7.0	14
S-32074	S-7	do	McHenry		B, C	100	99	99	92	79	13	--	26	7	A-4 (8)	15.0	113.0	9.0	42.5
S-32075	6900	Kansas	Grant		-------	100	97	94	77	69	25	19	29	9	A-4 (8)	16.5	106.7	10.5	87.5
S-32076	7520	Kentucky	Carter		-------	59	56	52	24	13	4	3	19	3	A-2-4 (0)	7.8	130.2	6.5	6
S-32078	7515	Indiana	Vanderburgh		-------	100	100	94	27	26	15	12	20	4	A-2-4 (0)	12.2	120.0	7.5	20
S-32082	7525	Missouri	Jackson		C	100	100	100	94	79	17	14	27	5	A-4 (8)	15.5	108.7	9.0	82
S-32083	7526	do	do		-------	100	100	100	94	80	15	13	28	6	A-4 (8)	15.7	108.0	9.5	85.5
S-32085	7514	Oklahoma	Okmulgee		A	92	86	85	36	25	15	10	27	7	A-4 (0)	12.0	117.6	7.0	32.5
Group B Soils																			
S-32572	7682	Colorado	Fremont		-------	67	52	39	29	27	16	10	26	9	A-2-4 (0)	11.8	124.2	11.0	14.5
S-32573	7687	Illinois	Will		B	100	100	99	97	93	45	37	47	25	A-7-6 (15)	22.0	100.3	15.0	143
S-32575	7695	Louisiana	Livingston		-------	100	99	98	94	86	17	12	26	4	A-4 (8)	17.7	104.4	15.5	33
S-32576	7701	Tennessee	Franklin		B	100	99	88	53	48	32	26	32	15	A-6 (6)	16.9	109.4	10.0	36
S-32577	7702	do	do		A	98	96	83	40	36	21	17	25	10	A-4 (1)	13.7	113.5	7.0	34
S-32578	7761	Montana	Silver Bow		-------	96	83	46	22	20	10	9	30	9	A-2-4 (0)	11.8	119.5	7.5	34
S-32579	7762	Michigan	Calhoun		A, B	92	87	75	37	31	13	9	23	8	A-4 (0)	7.9	115.9	10.0	37
S-32582	7776	Kansas	Douglas		-------	100	100	100	97	85	16	14	29	5	A-4 (8)	21.0	97.0	12.0	78
S-32583	7777	do	do		-------	100	100	100	98	86	16	14	29	6	A-4 (8)	18.5	102.2	12.0	73.5
S-32584	7779	Texas	Wichita		-------	55	39	25	16	13	5	5	25	9	A-2-4 (0)	8.4	129.0	7.0	18

[1] Data furnished by the Portland Cement Association.

weigh to 0.0002 gram on an analytical balance.

(2) Add 10 ml. of a dilute (2 percent) water solution of glycerol to the sample, and swirl the container gently to mix the contents.

(3) Heat at 110° C. (±3°) in a mechanical-convection oven containing a supply of glycerol to provide a source of free glycerol vapor in the oven chamber. Under these conditions, glycerol in excess of a monomolecular layer and water are both removed.

(4) Reweigh after equilibrium has been attained, normally after overnight heating. The gain in weight over the original oven-dry weight of the sample is due to the monomolecular layer of glycerol adsorbed on both the internal and external surfaces. The adsorbed glycerol is expressed as a percentage of the 110° C. dry weight of the soil.

(5) A distinction must be made between that portion of the glycerol retained on external surfaces of all clay minerals and that retained on internal surfaces of expanding minerals such as montmorillonite and vermiculite. On the internal surfaces both the top and bottom of the monomolecular layer of glycerol are in contact with clay surfaces. On the outside of the particles, however, only one side of the monomolecular layer is in contact with the clay surface. Therefore, a given amount of glycerol on internal surfaces accounts for the twice as much clay surface area as the same amount would if it were on external surfaces.

To make this distinction, a second determination is required. This is accomplished by determining the percentage of glycerol retained by replicate samples previously heated to 600° C., the glycerol retention being determined by the same procedure previously described. Heating to 600° C. normally collapses and irreversibly closes the internal spaces and thus renders them inaccessible to glycerol molecules. The difference between the original percentage of glycerol retained and that retained after heating to 600° C. is attributable to internal surfaces; the percentage measured after this preliminary heating is due to external surfaces only.

Based on X-ray diffraction evidence concerning the thickness of a monomolecular layer of glycerol, it has been shown (*5*) that one-hundredth of a gram of glycerol covers 35.3 square meters of internal clay surfaces; thus a glycerol retention of 1 percent on internal surfaces corresponds to 35.3 m.²/g. Similar deductions indicate that a retention of 1 percent of glycerol on external surfaces corresponds to a specific surface of 17.65 m.²/g.

For the soils used in this study, the surface area value of the whole soil was computed by multiplying the surface area found for the passing 40-mesh fraction by the percentage of the whole soil which passes the 40-mesh sieve. The surface area of the particles coarser than 40 mesh is so small as to be negligible. A hypothetical example of these computations follows:

	Percent
Glycerol retention of passing 40-mesh fraction	3. 50
Glycerol retention of same after preliminary 600° C. heating	1. 50
Retention due to external surface	1. 50
Retention due to internal surface (3.50−1.50)	2. 00

The indicated surface area of the passing 40-mesh fraction was computed as follows:

External surface	1.50	x	17.65	=26.5 m.²/g.
Internal surface	2.00	x	35.3	=70.6 m.²/g.
Total				97.1 m.²/g.

The percentage of the whole soil passing the 40-mesh sieve was 65 percent. Thus, the surface area of the whole soil would be 97.1 m.²/g. x 0.65=63.1 m.²/g. For this study the figure 63.1 would be rounded to the nearest one-half square meter per gram, or 63 m.²/g.

Cement requirements

The cement requirement determinations were performed by the staff of the P. C. A. Soil Cement Laboratory, using the method described in their *Soil Cement Laboratory Handbook, 1956*. Briefly, this method is as follows:

(1) Determine the grain-size distribution and Atterberg limits of the soil.

(2) Determine the moisture-density relations of a mixture of the soil and an assumed percentage of cement.

(3) Mold durability test specimens at optimum moisture and at cement contents thought to bracket the cement requirement, and test through 12 cycles of freezing and thawing. (Wet-dry tests may also be made, but they were not used for the soils of this investigation.) For A-1, A-2-4, and A-2-5 soils, the cement requirement is specified by the P. C. A. as that cement content at which test specimens lose 14 percent of their weight during the 12 cycles and the accompanying brushing procedure. For A-2-6, A-2-7, A-4, and A-5 soils, the loss permitted is 10 percent; for A-6 and A-7 soils, it is 7 percent. These loss criteria are based on information from a great many laboratory tests, the performance of field projects, and outdoor exposure of several thousand specimens.

(4) Check the estimated cement factor by molding and testing small specimens for compressive strength to insure that adequate hardening takes place at this cement content.

(5) For reporting and for field use, the cement factor is converted from a weight basis to a volume basis by the use of the following relation:

$$\text{Percent cement by volume} = \frac{D - \frac{D}{C}}{94} \times 100$$

Where:

D = Oven-dry density of the soil-cement specimen in lb. per cu. ft.

C = 100 plus the percent cement by weight of the oven-dry soil, the quantity divided by 100.

(6) The final recommended cement content is based to some extent on the judgment of the testing engineer. For example, the cement content indicated by the durability test data might be in a critical range; that is, where a small decrease in cement content would lead to very much higher than allowable freeze-thaw losses. In such a case, inadequate mixing on the job could result in an unsatisfactory product, and to insure against this, the testing engineer would recommend a slightly higher overall cement content than that provided for by the durability test data.

Results and Discussion

Surface area values and cement requirements for the group A samples (Nos. S-32044 to S-32085) are listed in table 1. The surface areas range from 6 to 90 m.²/g. The cement requirement values quoted (in terms of percent by volume) are those actually recommended for construction by the P. C. A. laboratory. Analysis shows that a statistically significant correlation [4] exists, $r=0.77$, significant at the 0.1 percent level. The following regression equation was derived from the data: Cement requirement=0.06 (surface area) +6.5. The standard error of estimate from this equation is 1.37 percent cement by volume. A plot of the relation is given in figure 1.

The statistical significance level of the correlation coefficient (0.1 percent) clearly indicates that a correlation actually exists. Nevertheless, the degree of correlation indicated by the correlation coefficient (0.77) is not strong enough to permit accurate predictions of cement requirement directly from surface area measurements.

Assuming the validity of the hypothesis that the surface area should be intimately associated with the cement need, the following possible reasons for not obtaining the expected closer correlation could be deduced:

(1) The surface area values used are based on the weight of the soil, surface area being expressed in square meters per gram, whereas the cement requirements are expressed on a volume basis. This naturally would weaken the correlation because there can be no overall relation between values expressed by weight and those expressed by volume, due to the variation in the densities of the different soil-cement products.

(2) As noted earlier, the cement requirement determined by the P. C. A. laboratory is based on the cement content at which maximum losses of 7, 10, or 14 percent occur during the 12-cycle freeze-thaw test; the limit applying depends on the AASHO classification of the soil. Two soils containing different clay minerals might have identical surface areas, but have greatly different grain-size distributions and plasticity indexes, thereby falling into different classifications. Accordingly, the cement requirement of one soil might be based on the 7-percent loss limit and that of the other on the 14-percent loss limit, despite their identical surface areas. In effect, this inserts a bias in the cement requirement values which is not correspondingly reflected in the surface area, and so tends to weaken the correlation.

(3) The cement requirements recommended are not derived solely from freeze-thaw loss data, but, as noted earlier, may be modified somewhat by engineering judgment. This added factor, while perfectly justified from a practical standpoint, is not related to a phys-

[4] See appendix on page 69 for definitions of statistical terms.

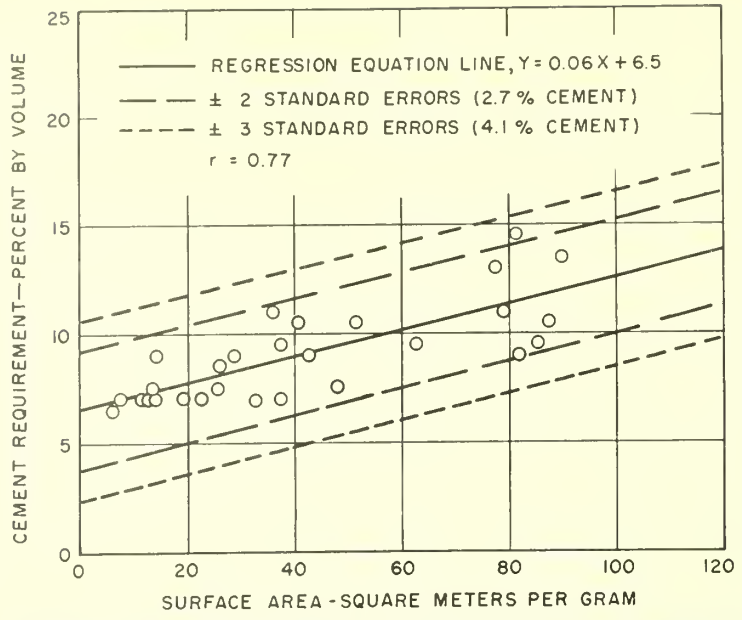

Figure 1.—Cement requirement recommended by the Portland Cement Association related to the surface area of group A soils.

ical measurement such as surface area and thus also tends to weaken the correlation.

(4) The cement requirement data are rounded to the nearest one-half percent of cement. Since the range in cement requirements encountered with these soils (6.5 to 14.5 percent) is only 16 times this figure, the rounding, while again justifiable from the practical standpoint, tends to weaken the correlation.

Cement requirement at 10-percent loss

It was thought that expressing the cement requirement in a different way would obviate these difficulties. Strictly for purposes of correlation, in place of the recommended cement requirement, it is proposed to use for all soils that cement content (in percent by weight) at which a 10-percent loss occurs in the freeze-thaw test. This places the cement content on a weight basis as is the surface area. The procedure eliminates the bias due to different limits of allowable loss for different soil groups by placing all of the soils on a uniform basis of 10-percent loss; it eliminates personal judgment factors; and when the cement contents are expressed to the nearest

Table 2.—Test data for 12 cycles of freezing and thawing, and data derived therefrom

B. P. R. soil No.	Test data furnished by the Portland Cement Association								Derived data			
	Specimen 1		Specimen 2		Specimen 3		Specimen 4		Cement content at 10-percent loss	Loss allowed in freeze-thaw test	Cement content at which allowed loss occurs	
	Cement content	Loss in weigbt	Cement content	Loss in weigbt	Cement content	Loss in weight	Cement content	Loss in weight			Percent by weight	Percent by volume
	GROUP A SOILS											
	Pct. by weight	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*	*Pct. by weight*	*Pct.*		
S-32044	5	13	7	5	--------	--------	--------	--------	5.8	14	4.8	5.7
S-32045	10	13	12	10	14	6	--------	--------	12.0	7	13.5	13.2
S-32046	6	13	8	9	10	7	--------	--------	7.5	7	10.0	10.4
S-32047	8	10	10	5	12	4	--------	--------	8.0	7	9.2	9.7
S-32050	9	48	11	42	13	33	15	30	[1]	10	[1]	[1]
S-32051	9	36	11	8	13	5	--------	--------	10.8	10	10.8	10.9
S-32053	7	23	9	2	--------	--------	--------	--------	8.2	10	8.2	8.8
S-32055	5	13	7	8	--------	--------	--------	--------	6.0	14	[2] 4.5	[2] 5.4
S-32056	8	35	10	7	12	6	14	5	9.6	10	9.6	10.5
S-32060	3	18	5	6	7	1	--------	--------	4.3	14	3.6	5.0
S-32061	5	3	7	2	9	2	--------	--------	[1]	14	[1]	[1]
S-32062	12	25	14	14	16	4	--------	--------	14.8	7	15.3	13.7
S-32063	8	13	10	10	12	8	--------	--------	10.0	7	[2] 12.9	[2] 12.6
S-32064	3	20	5	2	--------	--------	--------	--------	4.0	14	3.6	4.8
S-32065	4	100	6	11	8	7	--------	--------	6.5	14	5.9	7.0
S-32066	8	6	10	5	--------	--------	--------	--------	[1]	10	[1]	[1]
S-32067	10	7	12	6	14	5	--------	--------	[1]	10	[1]	[1]
S-32068	5	20	7	15	--------	--------	--------	--------	[1]	14	[2] 7.4	[2] 8.7
S-32069	6	7	8	5	--------	--------	--------	--------	[1]	10	[1]	[1]
S-32070	4	18	6	9	8	5	--------	--------	5.7	10	5.7	6.8
S-32071	7	13	9	5	11	4	--------	--------	7.7	10	7.7	8.8
S-32072	3	18	5	9	--------	--------	--------	--------	4.9	10	4.9	6.3
S-32074	6	5	8	3	10	3	--------	--------	[1]	10	[1]	[1]
S-32075	10	10	12	6	14	5	--------	--------	10.0	10	10.0	10.3
S-32076	3	17	5	6	--------	--------	--------	--------	4.3	14	3.4	4.8
S-32078	4	24	6	13	--------	--------	--------	--------	[1]	14	5.8	7.0
S-32082	7	11	9	7	11	6	--------	--------	7.5	10	7.5	8.1
S-32083	7	19	9	10	11	6	--------	--------	9.0	10	9.0	9.5
S-32085	5	11	7	7	--------	--------	--------	--------	5.7	10	5.7	6.8
	GROUP B SOILS											
S-32572	7	44	9	9	--------	--------	--------	--------	8.9	14	8.6	10.4
S-32573	11	13	14	8	17	5	--------	--------	12.8	7	15.0	13.9
S-32575	13	22	15	13	17	4	--------	--------	15.6	10	15.6	15.0
S-32576	8	8	10	6	12	5	--------	--------	[1]	7	9.0	9.5
S-32577	6	10	8	8	10	5	--------	--------	6.0	10	6.0	6.8
S-32578	4	21	6	14	8	6	--------	--------	7.0	14	6.0	7.2
S-32579	8	12	10	8	--------	--------	--------	--------	9.0	10	9.0	10.1
S-32582	13	7	16	4	19	2	--------	--------	[1]	10	[1]	[1]
S-32583	11	12	14	4	17	2	--------	--------	11.7	10	11.7	11.3
S-32584	3	28	5	4	--------	--------	--------	--------	4.5	14	4.2	5.5

[1] Not available from data supplied. [2] Obtained by extrapolation.

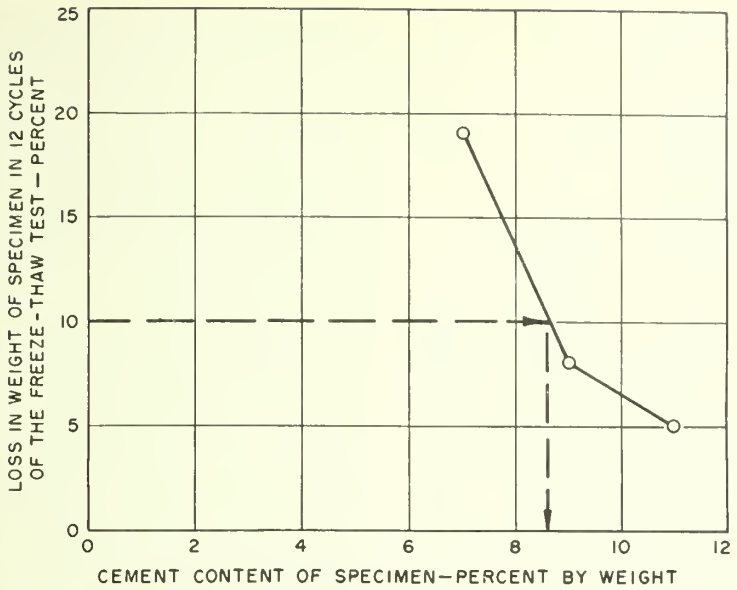

Figure 2.—Method of determining cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test. (In this illustration, the required value is 8.6 percent.)

0.1 percent, it eliminates bias due to excessive rounding of the values.

This expression of cement requirement was obtained by the procedure illustrated in figure 2. For the several freeze-thaw specimens prepared from the same soil at various cement contents, a plot was made relating the actual test loss to cement content, and the points were connected by straight lines. The cement content at which a 10-percent loss would occur was then read directly from this plot, by interpolation if necessary. The actual loss data and corresponding cement contents for all soils are given in table 2.

The relation between this new cement factor and surface area was investigated, with results as shown in figure 3. The analysis showed a distinct and marked improvement in the degree of correlation existing, the correlation coefficient r being 0.94, which demonstrates both the validity of the original hypothesis that cement requirement and surface area are closely related, and also that the disturbing factors previously discussed have been largely avoided by this method of estimating the required cement content.

The regression equation appropriate for predicting the cement content by weight at which a 10-percent loss occurs was calculated as $Y=0.076$ (surface area)$+3.93$. The standard error of estimate from this equation is 0.74 percent cement. It should be noted that for several of the samples, the available freeze-thaw test data could not be used to obtain the test value of the cement content at which a 10-percent loss would occur, except by questionable extrapolation; these samples have accordingly not been included in this correlation. In addition, three samples for which data were available (Nos. S–32056, S–32062, and S–32082) did not fit the correlation. Since these were more than three standard errors from the regression equation line, it is statistically valid to discard them from consideration on the grounds that they presumably do not belong to the same statistical population as the remainder of the samples. These three samples will be discussed later.

A sufficiently close correlation has thus been established to permit prediction of the cement content at which a 10-percent loss occurs from surface area measurements. It now remains to develop a procedure to convert the cement content, so predicted for an individual soil, back to an estimated cement requirement based on the specific freeze-thaw loss allowable for soils of its class. For those soils where a maximum of 10-percent loss is allowed (A–2–6, A–2–7, A–4, and A–5 soils), no adjustment of the predicted value is of course necessary; for those soils (A–6, A–7) having a maximum allowable loss of 7 percent, the predicted cement content would be increased; similarly for soils (A–1, A–2–4, A–2–5) where loss up to 14 percent is allowed, the predicted cement content would be decreased. By examination of the loss data, the appropriate corrections were estimated to be $+2.0$ percent and -0.7 percent, respectively. The corrected cement contents by weight can then be converted to a predicted cement requirement by volume through use of the formula previously listed.

The procedure was followed to obtain predicted cement requirements in percent by volume, which were then compared with the cement requirement (by volume) computed directly from the freeze-thaw test results. Agreement for group A samples (Nos. S–32044 through S–32085) was only reasonably good. The coefficient of correlation between the predicted and the test cement requirements was $r=0.87$, and the standard error of estimate was 1.3 percent cement.

Summarizing the results to this point, it has been shown that the following conditions prevailed:

(1) A definite though not very precise correlation exists between surface area and the cement requirement (percent cement by volume) actually recommended by the P. C. A. for this first group of plastic soils.

(2) The correlation was greatly improved by using, instead of the recommended cement requirement by volume, the cement content

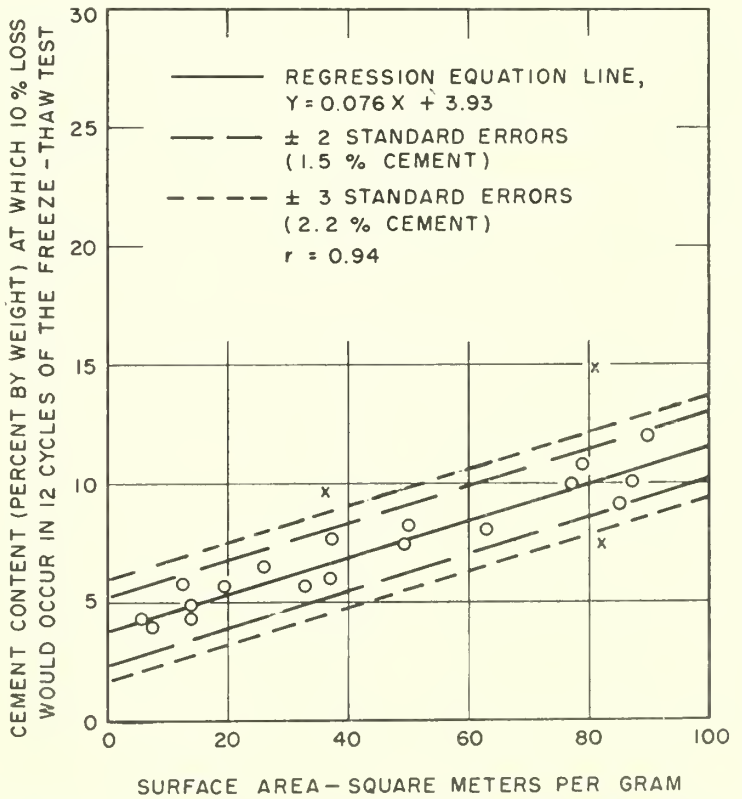

Figure 3.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of group A soils.

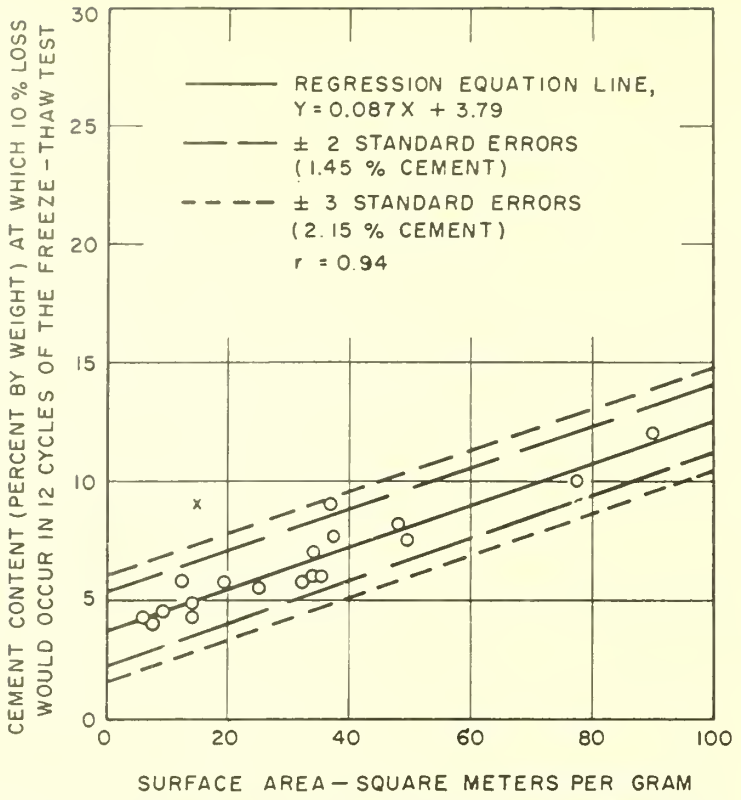

Figure 4.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of soils having less than 45 percent silt.

by weight at which an arbitrary figure of 10-percent loss would occur in 12 cycles of the freeze-thaw test. The computed regression equation permitted satisfactory predictions of this value to be made from the surface area.

(3) A procedure was developed for correcting this cement factor to predict the cement content (by weight) at which a 7- or 14-percent loss would occur, and by use of a given formula involving the density of the soil-cement product, this prediction could be converted to a volume basis.

(4) Predictions thus made were in reasonable agreement with cement requirements derived directly from the freeze-thaw test data, the correlation between the two sets of values being 0.87.

Another group of soils (sample Nos. S–32572 through S–32584), received after the work on the first group had been completed, provided an opportunity to check the validity of these results. These samples are described in table 1 as group B, and cement requirement test data for them are listed in table 2. After the surface areas were determined for these soils, predictions of cement requirement (by volume) were computed, using the regression equation derived for the soils of group A and the additional procedure just outlined. A comparison of these predicted values with cement requirements computed directly from the freeze-thaw test data indicated good agreement for some of the samples but considerable deviations for others.

Upon examination of the engineering test data for group B soils, it was noted that all ut one of the deviant samples were very igh in silt content, silt being taken as that portion passing the 200-mesh sieve and coarser than 0.005 mm. When the data for the samples of group A were reexamined, it was found that here also high silt content was associated with relatively poor agreement between predicted and test results. In particular, it was noted that the three soils which were discarded from the previous correlation were high in silt content. For such soils, the cement requirement is evidently governed by some property or properties other than surface area.

Influence of silt content

In order to verify this premise, the data for both groups of soils taken together were divided into two categories on the basis of silt content. Upon examination of the data, an appropriate dividing line appeared to be at a silt content of 45 percent. The correlation between surface area and the cement content (by weight) at which a 10-percent loss occurred in the freeze-thaw test was recomputed separately for each category. The category (22 samples) with silt content under 45 percent showed a high degree of correlation ($r=0.94$) similar to that previously determined for the first batch alone.

One deviant sample (No. 32572) did not fit the correlation, and since it was considerably more than three standard errors from the regression equation line, it was discarded. This sample is discussed later. On the other hand, the category consisting of soils with silt contents of 45 percent or higher showed essentially zero correlation ($r=-0.03$). The data for the two categories of samples are plotted separately in figures 4 and 5. The striking difference in correlation coefficients demonstrates clearly that the two sets of samples represent different populations. For the soils of lower silt content, the cement requirement is essentially a function of surface area, as has previously been determined; but for soils of higher silt content, there is little relation between the two.

The regression equation was then calculated for the soils of low silt content. It differs only slightly from the corresponding equation previously calculated for group A samples alone. The new equation for cement content (by weight) at which a 10-percent loss occurs is: $y=0.087$ (surface area)$+3.79$. The equation and the previously described procedure were then used to predict the values of cement requirement for these soils. These predictions are compared in the upper part of table 3 with the cement requirements derived from the freeze-thaw test data. Agreement between the two sets of values is good (correlation coefficient $r=0.92$, standard error of estimate $=0.9$ percent cement), again with the exception of sample No. S–32572 previously noted as being a deviant sample. Neglecting this sample, the average deviation between predicted and test cement requirements was only 0.6 percent cement. Among the remaining

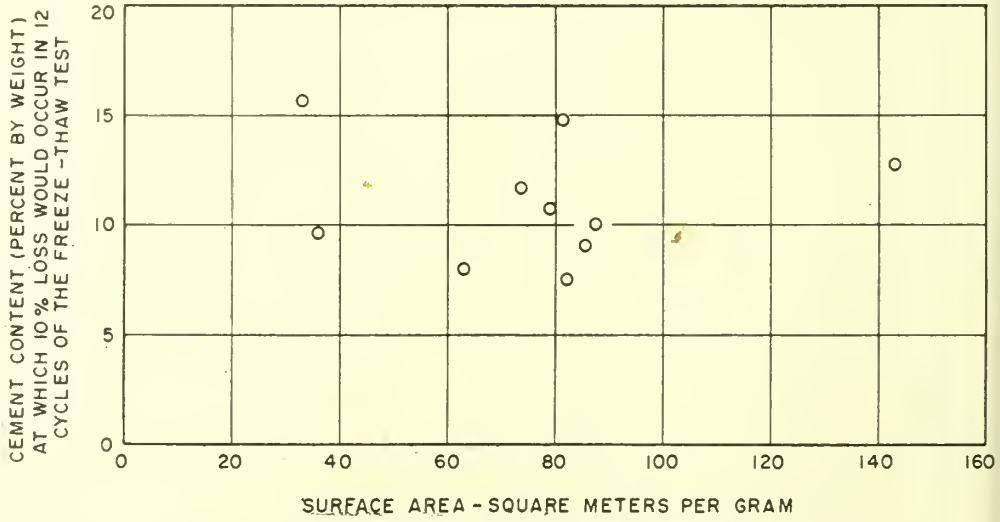

Figure 5.—Cement content at which a 10-percent loss would occur in a 12-cycle freeze-thaw test related to the surface area of soils having 45 percent or more silt.

21 soils, 6 were classified in the highly plastic AASHO groups A–2–6, A–2–7, A–6, and A–7. For all six soils, predictions were within 0.2 percent cement of the test value.

Clay mineral examination

The clay minerals present in the various samples were identified by X-ray diffraction techniques. The clay fractions were nearly all mixtures of two or more clay minerals, including among them montmorillonite, illite, kaolinite, chlorite, and vermiculite. Most contained at least a detectable amount of montmorillonite, and several were almost pure montmorillonite. None of the clays was principally illite or principally kaolinite, although several consisted largely of a chlorite-like clay mineral. From the evidence available, it seems that, in general, the cement requirement is influenced by the surface area itself, without regard to the specific type of clay mineral from which the surface area is derived.

No definite explanation has been found for the behavior of sample No. S–32572 (point X in figure 4), previously noted as a deviant sample. The actual cement requirement is much higher than the predicted value. X-ray diffraction examination indicated that this soil consists primarily of finely divided calcium carbonate (caliche), much of it in the clay size range. Further study is needed to determine the effect of this type of material on the cement requirement.

From the results obtained, a procedure for predicting cement requirements of plastic soils of less than 45 percent silt content was formulated. This is diagramed in figure 6. For soils to which it can be applied, the procedure eliminates the time-consuming freeze-thaw tests. It would still be necessary, however, to prepare small specimens at and near the predicted cement requirement, and test them for compressive strength (or by other suitable means) to insure that adequate hardening was actually taking place. Furthermore, the predicted cement content should be modified by appropriate engineering judgment to compensate for factors such as difficulty of adequate mixing in the field and possible local variations in the soil materials.

It is recognized that these findings are based on only a limited number of samples, and are not necessarily applicable to all soils. For example, other authors have reported that certain types of soil organic matter strongly influence the cement requirement. Although a number of the soils in this study were moderately high in organic matter, there was no evidence of appreciable effects on cement requirement. With a larger group of samples, soils containing such deleterious organic matter might have been encountered.

Table 3.—Comparison of cement requirements predicted from surface area data with cement requirement from freeze-thaw test data

B. P. R. soil No.	AASHO classification	Surface area	Prediction of cement content at which 10-percent loss occurs [1]	Corrections for soils where 7- or 14-percent loss is allowed	Corrected prediction of cement requirement		Cement requirement computed from test data	Deviation (column 7 minus column 8)
					Percent by weight	Percent by volume		
Category 1: Plastic Soils With Less Than 45 Percent Silt Content								
		m.²/g.	*Percent by weight*	*Percent by weight*			*Percent by volume*	*Percent by volume*
S–32044	A–2–4(0)	12.5	4.9	−0.7	4.2	4.9	5.7	−0.8
S–32045	A–6 (7)	90	11.6	+2.0	13.6	13.1	13.2	−.1
S–32046	A–6 (2)	49.5	8.1	+2.0	10.1	10.6	10.4	+.2
S–32053	A–2–6 (1)	48	8.0	----------	8.0	8.6	8.8	−.2
S–32055	A–4 (8)	37.5	7.1	−.7	6.4	7.6	5.4	+2.2
S–32060	A–1–b (0)	14	5.0	−.7	4.3	5.9	5.0	+.9
S–32063	A–7–6 (14)	77.5	10.5	+2.0	12.5	12.4	12.6	−.2
S–32064	A–1–b (0)	75	4.4	−.7	3.7	5.0	4.8	+.2
S–32065	A–2–4 (0)	25.5	6.0	−.7	5.3	6.5	7.0	−.5
S–32068	A–2–4 (0)	28.5	6.3	−.7	5.6	6.8	8.7	−1.9
S–32070	A–4 (1)	19.5	5.5	----------	5.5	6.6	6.8	−.2
S–32071	A–4 (4)	37.5	7.1	----------	7.1	8.5	8.8	−.3
S–32072	A–2–6 (0)	14	5.0	----------	5.0	6.4	6.3	+.1
S–32076	A–2–4 (0)	6	4.3	−.7	3.6	4.8	4.8	.0
S–32078	A–2–4 (0)	20	5.5	−.7	4.8	5.8	7.0	−1.2
S–32085	A–4 (0)	32.5	6.6	----------	6.6	7.7	6.8	+.9
S–32576	A–6 (6)	36	6.9	+2.0	8.9	9.5	9.5	.0
S–32577	A 4 (1)	34	6.8	----------	6.8	7.7	6.8	+.9
S–32578	A–2–4 (0)	34	6.8	−.7	6.1	7.3	7.2	+.1
S–32579	A–4 (0)	37	7.0	----------	7.0	8.1	10.1	−2.0
S–32584	A–2–4 (0)	9	4.6	−.7	3.9	5.1	5.5	−.4
S–32572 [2]	A–2–4 (0)	14.5	5.1	−.7	4.4	5.6	10.4	−4.8
Category 2: Plastic Soils With 45 Percent or More Silt Content								
S–32047	A–6 (9)	63	9.3	+2.0	11.3	11.4	9.7	+1.7
S–32051	A–4 (8)	79	10.7	----------	10.7	10.8	10.9	−.1
S–32056	A–4 (8)	36	6.9	----------	6.9	7.7	10.5	−2.8
S–32062	A–7–6 (11)	81	10.8	+2.0	12.8	11.7	13.7	−2.0
S–32075	A–4 (8)	87.5	11.4	----------	11.4	11.6	10.3	+1.3
S–32082	A–4 (8)	82	10.9	----------	10.9	11.4	8.1	+3.3
S–32083	A–4 (8)	85.5	11.2	----------	11.2	11.6	9.5	+2.1
S–32573	A–7–6 (15)	143	16.2	+2.0	18.2	16.2	13.9	+2.3
S–32575	A–4 (8)	33	6.7	----------	6.7	7.1	15.0	−7.9
S–32583	A–4 (8)	73.5	10.2	----------	10.2	9.8	11.3	−1.5

[1] Determined by regression equation.
[2] Deviant sample discarded in computing the regression equation.

Appendix

The statistical terms employed in this study are as follows:

Correlation coefficient (r).—A term which indicates the degree of association or relation between the measured values of one property and the corresponding measured values of another property, for a specified group of samples. This term varies from 1.0, indicating that a perfect functional relation exists and that one property could be predicted with absolute accuracy from knowledge of the other, to zero, which indicates a complete lack of relation between the two properties. If the relation is *direct*, i. e., if one property increases with an increase in the other, the correlation coefficient is *positive;* if the relation is *inverse* (one property decreases with an increase in the other), the correlation coefficient is *negative.* Generally, a correlation coefficient above 0.9 is required for the correlation to be good enough to permit predictions of one value from the other with a reasonable degree of accuracy.

Statistical significance level of correlation coefficient.—This is a measure of the probability that so large a correlation coefficient as has been computed from the data could arise by pure chance sampling from a population in which there is in fact no correlation. A 0.1-percent or even a 1-percent significance level indicates that a correlation almost certainly does exist.

Regression equation.—If a linear correlation exists between two properties of a group of samples, and a plot is made of property Y versus property X for all samples of the group, an array of scattered points results. A straight line may be drawn through the scattered points in such a way that it best fits the data, using as the criterion of "best fit" that the sum of the squares of the deviations of all of the points from the line is at a minimum. The equation of this line is called the regression equation, and its use permits the best estimate of values of property Y to be made from measured values of property X.

Standard error of estimate (Sy).—This is a measurement of deviation or degree of scatter of the points around the regression equation line. It has the same dimensions as the dependent variable, Y, and it provides an estimate of the uncertainty of the prediction of Y from X by means of the regression equation. If the normal distribution of errors holds, 19 out of 20 samples should fall within 2 standard errors of the regression equation line, and 997 out of 1,000 within 3 standard errors.

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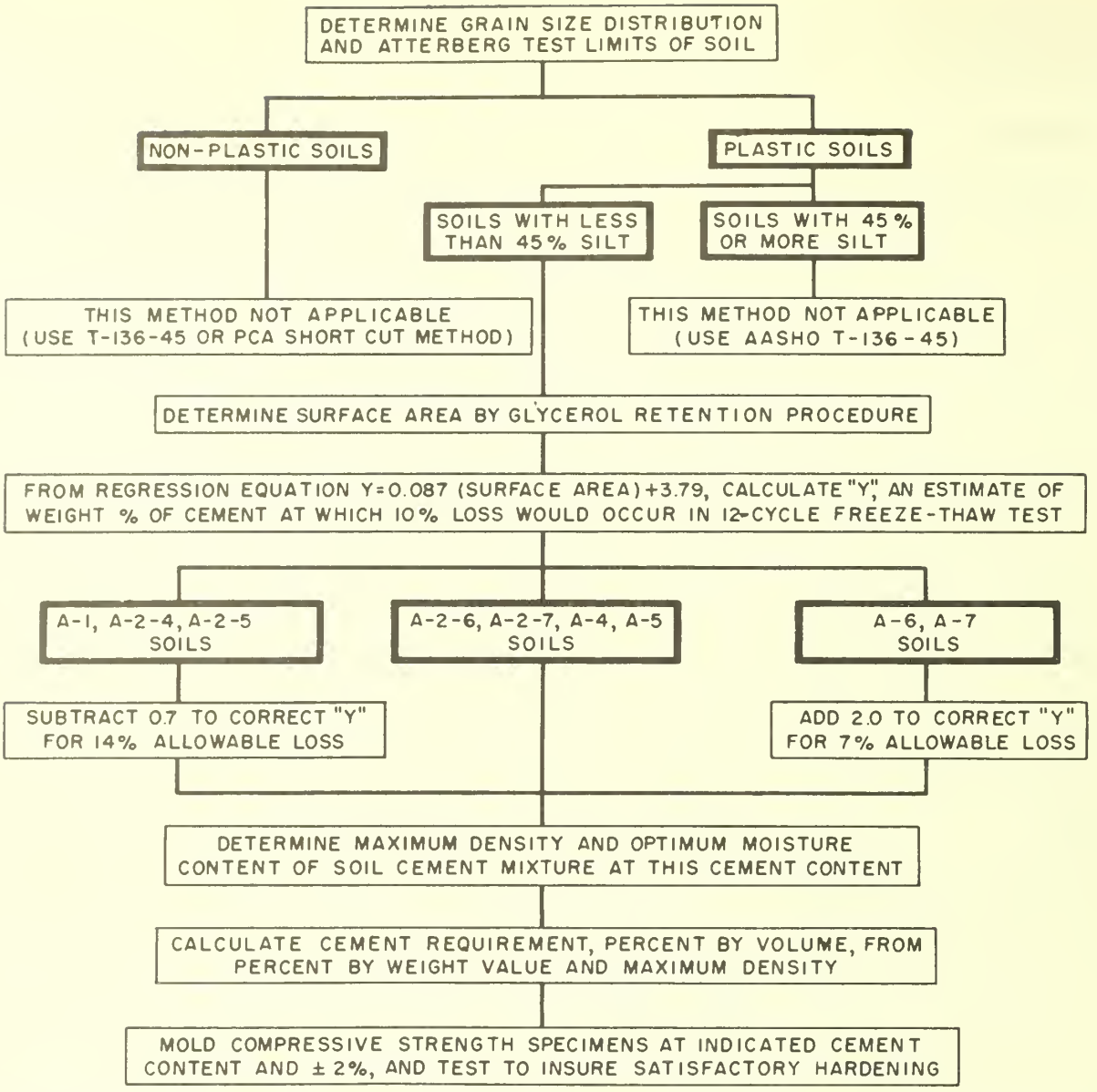
DETERMINE GRAIN SIZE DISTRIBUTION AND ATTERBERG TEST LIMITS OF SOIL
NON-PLASTIC SOILS
PLASTIC SOILS
SOILS WITH LESS THAN 45% SILT
SOILS WITH 45% OR MORE SILT
THIS METHOD NOT APPLICABLE (USE T-136-45 OR PCA SHORT CUT METHOD)
THIS METHOD NOT APPLICABLE (USE AASHO T-136-45)
DETERMINE SURFACE AREA BY GLYCEROL RETENTION PROCEDURE
FROM REGRESSION EQUATION Y=0.087 (SURFACE AREA)+3.79, CALCULATE "Y," AN ESTIMATE OF WEIGHT % OF CEMENT AT WHICH 10% LOSS WOULD OCCUR IN 12-CYCLE FREEZE-THAW TEST
A-1, A-2-4, A-2-5 SOILS
A-2-6, A-2-7, A-4, A-5 SOILS
A-6, A-7 SOILS
SUBTRACT 0.7 TO CORRECT "Y" FOR 14% ALLOWABLE LOSS
ADD 2.0 TO CORRECT "Y" FOR 7% ALLOWABLE LOSS
DETERMINE MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT OF SOIL CEMENT MIXTURE AT THIS CEMENT CONTENT
CALCULATE CEMENT REQUIREMENT, PERCENT BY VOLUME, FROM PERCENT BY WEIGHT VALUE AND MAXIMUM DENSITY
MOLD COMPRESSIVE STRENGTH SPECIMENS AT INDICATED CEMENT CONTENT AND ± 2%, AND TEST TO INSURE SATISFACTORY HARDENING

Figure 6.—Short-cut method, using surface area measurements, for determining cement requirement of plastic soils containing less than 45 percent silt.

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Lateral Placements of Trucks on Two-Lane Highways and Four-Lane Divided Highways

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported [1] by ASRIEL TARAGIN, Chief, Traffic Performance Branch

This article describes the driving habits of commercial-vehicle operators with respect to encroachment on the left traffic lane and on the shoulders of two-lane highways and four-lane divided highways. Lateral placement data were obtained at 119 rural locations in 17 States for 135,000 vehicles of all types of which more than 20,000 were commercial vehicles.

The positioning of vehicles in the traffic lane is particularly important to the highway engineer in designing the roadway to meet the requirements of the different classes of vehicles.

Both portland cement concrete and bituminous pavements were included in the study. Some pavement sections had grass or gravel shoulders; others had bituminous-paved shoulders; and still others, a combination of bituminous materials and gravel.

On two-lane highways with 12-foot traffic lanes and grass or gravel shoulders, only 0.8 percent of the trucks were encroaching on the shoulders, however, 2.3 percent were encroaching on the left traffic lane. The widest range in vehicle placements was found on road sections with matching bituminous pavements and shoulders. On these sections, nearly 70 percent of all trucks were traveling partly on the shoulders.

Studies made on level sections of four-lane divided highways indicated that placements in the right traffic lane were similar to those found on two-lane (two-directional) highways. On horizontal curves, drivers tended to travel closer to the center of the pavement on four-lane divided highways than was the case for two-lane highways.

THE Bureau of Public Roads has been conducting research studies for a number of years for the purpose of determining the effects of various highway geometric design features on driver behavior. The almost universally accepted width of 12 feet for a traffic lane was based largely on the results of a comprehensive study of the effect of roadway width on vehicle operations.[2] Although the results of this study were reported in 1945, the basic data were collected prior to World War II. At that time there was a very limited mileage of roads with 12-foot lanes. Since then, nearly all newly constructed main highways have 12-foot traffic lanes. Phenomenal increases in traffic volumes and particularly in the sizes and weights of commercial vehicles have also taken place during this period.

Results of recent road test studies [3] have highlighted the importance of the positioning of trucks in the traveled lane. Current data on lateral positions of trucks in normal traffic were needed for designing new highways and for scheduling vehicle operations on the AASHO Road Test in Illinois. The Bureau of Public Roads was requested to supply such data which were to be representative of two-lane highway and four-lane divided highway operations on sections with 12-foot traffic lanes. Bituminous and portland cement concrete pavements were to be included in the study.

Vehicle placement data were already available as a result of a number of studies recently conducted as part of the regularly scheduled driver behavior research program.[4] Special studies were conducted during the summer of 1956 in Illinois, Missouri, and Ohio to supplement existing data and to include several States in the general area of the AASHO Road Test. The results presented are based on placement data recorded at 119 rural locations in 17 States, as shown in table 1. A total of 135,000 vehicles of all types were studied, of which more than 20,000 commercial vehicles are included in the present analysis.

Commercial vehicles for this study include trucks with dual tires on the rear axles, tractor-trailer combinations, and buses. Placement data were recorded for a minimum of 100 commercial vehicles at each location. For the purpose of this article commercial vehicles will be identified as "trucks." These trucks constituted on the average about 15 percent of the total traffic. The range in

Table 1.—Number of locations and trucks studied for placements on pavement sections with 12-foot traffic lanes

State	Year of study	Two-lane highways			Four-lane divided highways		
		Locations studied	Trucks		Locations studied	Trucks	
			Number observed	Percent of total traffic		Number observed	Percent of total traffic
Arizona	1955	9	896	15			
California	1955	10	2,848	15			
Colorado	1955	3	547	10			
Idaho	1955	7	861	11			
Illinois	1956	6	1,119	19	10	1,532	12
Iowa	1955	3	622	15			
Louisiana	1956	5	917	17	1	150	6
Maryland	1954	5	1,192	13			
Missouri	1956	6	651	13	8	865	14
New Mexico	1955	2	472	15			
Ohio	1956	4	702	22	5	703	21
Oregon	1955	5	838	12			
Pennsylvania	1948				13	2,615	14
Texas	1955	6	745	14			
Utah	1955	7	851	15			
Virginia	1953				2	935	20
Washington	1955	2	342	10			
Total		80	13,603	14	39	6,800	15

[1] Data were recorded in the field by William D. Whitby and Bernell A. Porter, engineering aids, Bureau of Public Roads, in cooperation with 17 State highway departments. Tabular and graphic material appearing in this issue were prepared under the direction of Robert E. Payne, statistical clerk, Bureau of Public Roads.

[2] *Effect of roadway width on vehicle operation*, by A Taragin. PUBLIC ROADS, vol. 24, No. 6, Oct.–Nov.–Dec. 1945.

[3] *Road Test One—MD.* Highway Research Board, Special Report 4, 1952, Washington, D. C.; also *The WASHO Road Test, Part 2, Test Data, Analyses, Findings.* Highway Research Board, Special Report 22, 1955, Washington, D. C.

[4] *Driver behavior related to types and widths of shoulders on two-lane highways*, by Asriel Taragin. PUBLIC ROADS, vol. 29, No. 9, Aug. 1957.

Table 2.—Distribution of placements of trucks on rural two-lane pavements with 12-foot traffic lanes and grass or gravel shoulders

Distance from center of truck to centerline of pavement [1]	Distribution of placements on level tangent sections paved with—		Distribution of placements on level curve sections (2° to 6°) paved with—				All pavement sections
	Portland cement concrete	Bituminous materials	Portland cement concrete [2]		Bituminous materials [2]		
			Inside lane	Outside lane	Inside lane	Outside lane	
Feet	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
0.0–0.9	0.1	[3]	--------	--------	--------	--------	[3]
1.0–1.9	--------	0.2	--------	--------	0.5	0.2	0.2
2.0–2.9	.4	.4	--------	1.1	.2	1.7	.6
3.0–3.9	.7	1.2	2.1	--------	.4	4.2	1.5
4.0–4.9	6.5	11.6	16.6	9.1	4.7	20.3	11.5
5.0–5.9	28.2	30.8	31.2	33.6	18.8	39.7	30.4
6.0–6.9	47.2	41.8	38.5	47.4	50.4	27.8	42.2
7.0–7.9	15.5	11.1	11.6	8.8	24.0	5.6	12.8
8.0–8.9	1.4	1.8	--------	--------	.7	.5	.7
9.0–9.9	--------	.5	--------	--------	.3	--------	.1
10.0–10.9	--------	.1	--------	--------	--------	--------	[3]
11.0–11.9	--------	.3	--------	--------	--------	--------	[3]
12.0–12.9	--------	.2	--------	--------	--------	--------	[3]
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0

[1] For distances below 4 feet, trucks were straddling the centerline; for distances greater than 8 feet, trucks were encroaching on the shoulder.
[2] Inside lanes on curves to the right and outside lanes on curves to the left. [3] Less than 0.05 percent.

traffic volume was from 100 to 500 vehicles per hour for both directions of travel on two-lane roads, and from 200 to 1,000 vehicles per hour for one direction of travel on four-lane divided highways.

Truck placement data were recorded on bituminous and on portland cement concrete surfaces, on level tangent sections, and on horizontal curves of 2 to 6 degrees. There were sections with grass or gravel shoulders for all four of these conditions. In addition, there were bituminous-paved shoulders adjacent to the travel lane on some of the two-lane level tangent sections.

For the two-lane studies each truck was classified as to whether it was free-moving (uninfluenced by other traffic), meeting other vehicles (affected by opposing traffic), or other (neither free-moving nor meeting). Trucks overtaking and passing other vehicles were not included in any of these groups.

For the four-lane studies trucks were classified into three similar groups: The first group was free-moving trucks; the second group was identified as adjacent to other vehicles, in place of meeting other vehicles as used for two-lane roads; and the third group consisted of trucks that were not placed in either the first or second classification. It was found that because of the low volumes during the study periods the greatest percentage of trucks were in the free-moving group. The average lateral positions of the meeting trucks (on two-lane roads) and adjacent trucks (on four-lane roads) were 0.4 to 0.8 foot farther from the centerline of the pavement than the free-moving trucks. The distributions of placements for free-moving, for meeting or adjacent, and for other trucks have been combined in this article, and the results are shown in tables 2–5 and in figures 1 and 2.

The placement values shown in these tables and graphs are for the position of the center of the truck with respect to the center of the pavement. Placement data were recorded for both the right and left wheels of each vehicle. It was found, however, that the position of the center of the vehicle could be determined more accurately and the results were simpler to analyze. The width of trucks for this study was assumed to be 8 feet. Thus, to determine the position of the right edge of the right wheels with respect to the pavement edge on 12-foot lanes, the value for the center of the truck was subtracted from 8. This value of 8 is obtained by subtracting half the truck width from the lane width, or 12 feet minus 4 feet. For 11-foot lanes the factor is 7, and for 10-foot lanes the factor is 6.

Lateral Placements on Two-Lane Highways

Table 2 shows the distribution of placements of trucks on two-lane roads with grass or gravel shoulders. Of the 40 locations studied, 32 were on level tangent sections and 8 were on horizontal curves. It is evident in figure 1

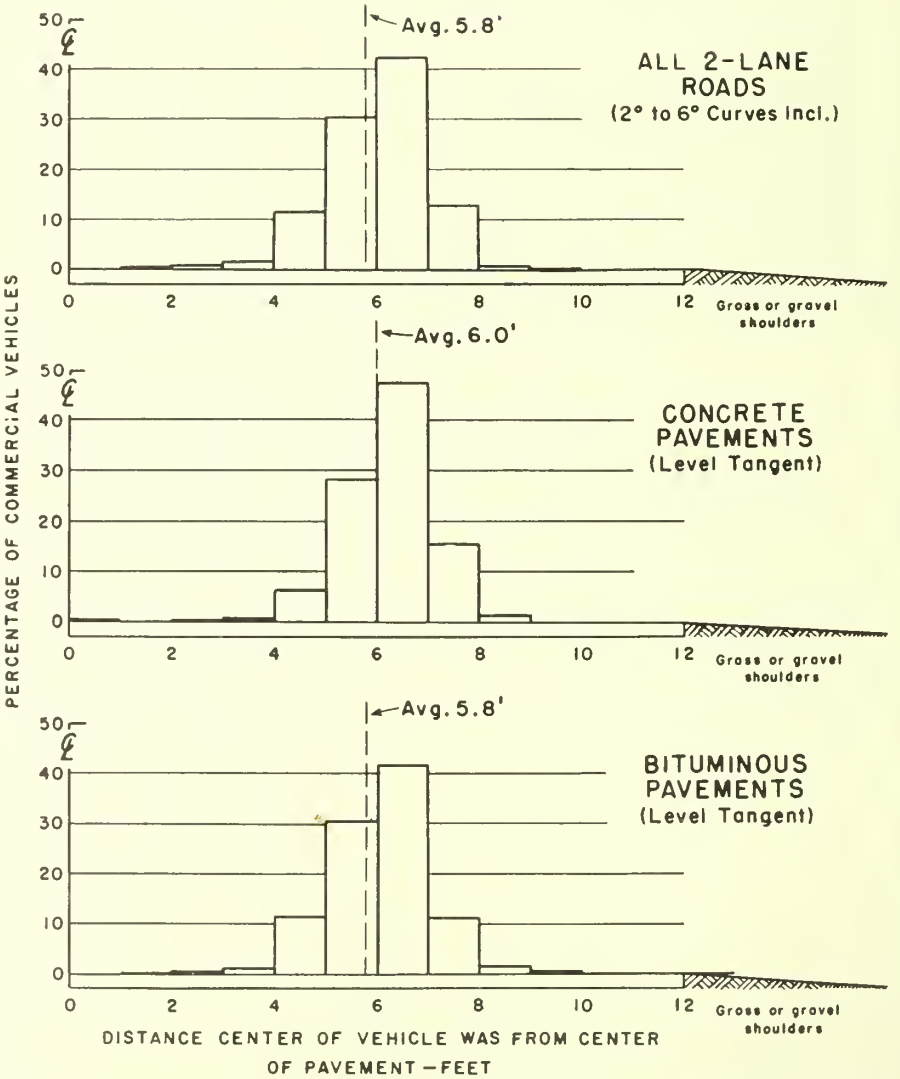

Figure 1.—Distribution of lateral positions of trucks on two-lane highways with grass or gravel shoulders.

Table 3.—Placement data of trucks observed at individual locations on rural two-lane level tangent pavements with 12-foot bituminous traffic lanes and grass or gravel shoulders

State	Study section number	Shoulder width, in feet	Number of trucks observed	Average placement,[1] in feet	Percentage of trucks—		
					Straddling centerline of pavement	Traveling entirely in traffic lane	Encroaching on shoulder
Iowa	1	2	90	5.5	--------	100.0	----------
Idaho	2	3	48	5.8	2.1	95.8	2.1
Colorado	3	4	210	5.6	2.9	93.3	3.8
Illinois	4	4	219	6.0	.9	99.1	----------
Missouri	5	4	107	5.7	.9	98.1	1.0
Arizona	6	4–6	165	6.1	--------	99.4	.6
Idaho	7	5	129	5.9	1.5	87.7	10.8
Missouri	8	5	119	5.6	--------	100.0	----------
Texas	9	5	181	6.0	--------	98.9	1.1
Maryland	10	5–7	254	5.8	1.2	98.8	----------
Louisiana	11	6	117	5.3	--------	98.3	1.7
Do	12	6	181	5.6	1.2	97.7	1.1
New Mexico	13	7	185	5.9	1.1	94.1	4.8
Louisiana	14	8	141	5.2	3.5	95.8	.7
Do	15	8	141	5.5	--------	100.0	----------
Maryland	16	8	289	5.7	7.2	92.8	----------
Do	17	8	259	6.1	.4	97.7	1.9
New Mexico	18	8	287	6.2	2.1	87.7	10.2
Utah	19	8	121	6.1	1.7	92.5	5.8
Colorado	20	10	170	5.9	1.8	90.5	7.7
Illinois	21	10	131	5.9	2.3	96.2	1.5
Do	22	10	211	5.9	1.9	95.4	2.7
Iowa	23	10	216	6.4	.5	92.6	6.9
Oregon	24	10	111	6.0	4.5	87.4	8.1
Washington	25	10	123	5.4	7.3	92.7	----------
Average or total	------	7	4,206	5.8	1.8	95.3	2.9

[1] Distance from center of truck to centerline of pavement.

hat on level tangent sections there was little ifference between the average placement on ituminous surfaces and concrete surfaces. he slightly greater range in the distribution of lacements on the bituminous pavements is due primarily to the larger sample size and the greater number of locations studied.

Horizontal curves of less than 6 degrees apparently did not affect the lateral placement of vehicles. This was particularly true on concrete surfaces. Because of the small difference between the placements on the several types of alinement on two-lane roads, the data for all sections were combined and are shown in the last column of table 2 and in the top graph of figure 1. Only 0.8 percent of all trucks encroached on the grass or gravel shoulders of two-lane roads with 12-foot traffic lanes. Encroachment on the left lane amounted to 2.3 percent. Nearly three-fourths of the trucks maintained a lateral position within 1 foot of the center of the lane. The average lateral position of the centers of all trucks was 5.8 feet from the centerline of the pavement on two-lane roads. In other words, the center of the average truck was 0.2 foot to the left of the center of the traffic lane.

Table 3 shows data for each of the 25 locations included in the results shown in the third column of table 2. At only 5 of the 25 locations did the lateral placements differ more than 0.3 foot from the average placement for all locations. Forty percent of the locations had placements within 0.1 foot of the average. It is obvious that the average data are very consistent and may be used with confidence.

Table 4 shows the distribution of placements on two-lane highways having 12-foot lanes with bituminous shoulders. Only level tangent sections are included. The widest range in the distribution of placements was found on sections with matching bituminous

Table 4.—Distribution of placements of trucks on rural two-lane level tangent pavements with 12-foot traffic lanes and bituminous-paved shoulders

Distance from center of truck to centerline of pavement[1]	Portland cement concrete and 8- to 10-foot bituminous shoulders	Bituminous-paved traffic lanes with—		
		Combination-type shoulders, 8 to 10 feet wide[2]	Contrasting bituminous shoulders, 6 to 10 feet wide	Matching bituminous shoulders, 8 feet wide
Feet	*Percent*	*Percent*	*Percent*	*Percent*
0.0–0.9	-----	0.1	0.2	0.1
1.0–1.9	1.6	2.8	.1	.1
2.0–2.9	.1	.6	.2	.3
3.0–3.9	.1	.8	.7	.7
4.0–4.9	6.6	3.8	4.4	1.7
5.0–5.9	23.1	18.2	14.6	5.3
6.0–6.9	47.5	34.3	31.3	9.7
7.0–7.9	12.5	18.5	18.3	13.4
8.0–8.9	5.0	11.6	13.3	13.1
9.0–9.9	.8	3.5	6.1	12.0
10.0–10.9	.6	1.9	3.6	14.7
11.0–11.9	1.1	1.4	2.4	10.6
12.0–12.9	.8	2.4	2.4	7.6
13.0–13.9	.2	.1	2.0	5.7
14.0–14.9	-----	-----	.1	2.8
15.0–15.9	-----	-----	.2	.9
16.0–16.9	-----	-----	.1	.7
17.0 and over	-----	-----	-----	.6
Total	100.0	100.0	100.0	100.0

[1] For distances less than 4 feet, trucks were straddling the centerline of the pavement; for distances greater than 8 feet, trucks were encroaching on the shoulder.
[2] Four-foot contrasting bituminous shoulders plus 4 to 6 feet of gravel on the outside.

Table 5.—Distribution of placements of trucks on rural four-lane divided highways with 12-foot traffic lanes and grass or gravel shoulders

Distance from center of truck to centerline of pavement[1]	Distribution of placements on level tangent sections paved with—		Distribution of placements on level curve sections (2° to 6°) paved with—			All pavement sections
	Portland cement concrete	Bituminous materials	Portland cement concrete		Bituminous materials, curve to left	
			Curve to right	Curve to left		
Left Traffic Lane						
Feet	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*	*Pct.*
8.0–8.9	0.1	0.1	----------	----------	----------	[2]
7.0–7.9	.5	1.0	0.2	0.7	1.9	0.9
6.0–6.9	2.1	3.2	1.6	3.6	3.1	2.7
5.0–5.9	1.5	.9	1.0	.9	1.6	1.2
4.0–4.9	.7	.4	.3	1.6	.8	.8
3.0–3.9	.3	.1	.5	1.0	.4	.5
2.0–2.9	.3	----------	----------	.6	.8	.2
1.0–1.9	.1	.2	.2	.3	.4	.3
0.0–0.9	.1	----------	.4	1.1	----------	.3
Total	5.7	5.9	4.2	9.8	9.0	6.9
Right Traffic Lane						
0.0–0.9	0.3	----------	----------	0.3	----------	0.1
1.0–1.9	.2	0.4	0.7	1.5	2.4	1.1
2.0–2.9	.5	----------	.8	5.3	2.2	1.8
3.0–3.9	2.2	1.1	11.9	9.1	8.7	6.6
4.0–4.9	13.1	6.4	22.8	29.7	19.2	18.2
5.0–5.9	37.4	24.8	40.9	30.2	29.6	32.6
6.0–6.9	36.2	49.3	17.2	12.2	23.1	27.6
7.0–7.9	4.2	11.3	1.5	1.9	5.8	4.9
8.0–8.9	.2	.6	----------	----------	----------	.2
9.0–9.9	----------	.2	----------	----------	----------	[2]
Total	94.3	94.1	95.8	90.2	91.0	93.1

[1] For distances below 4 feet, trucks were straddling the centerline; for distances greater than 8 feet, trucks were encroaching on the shoulders for right lane and median for left lane.
[2] Less than 0.05 percent.

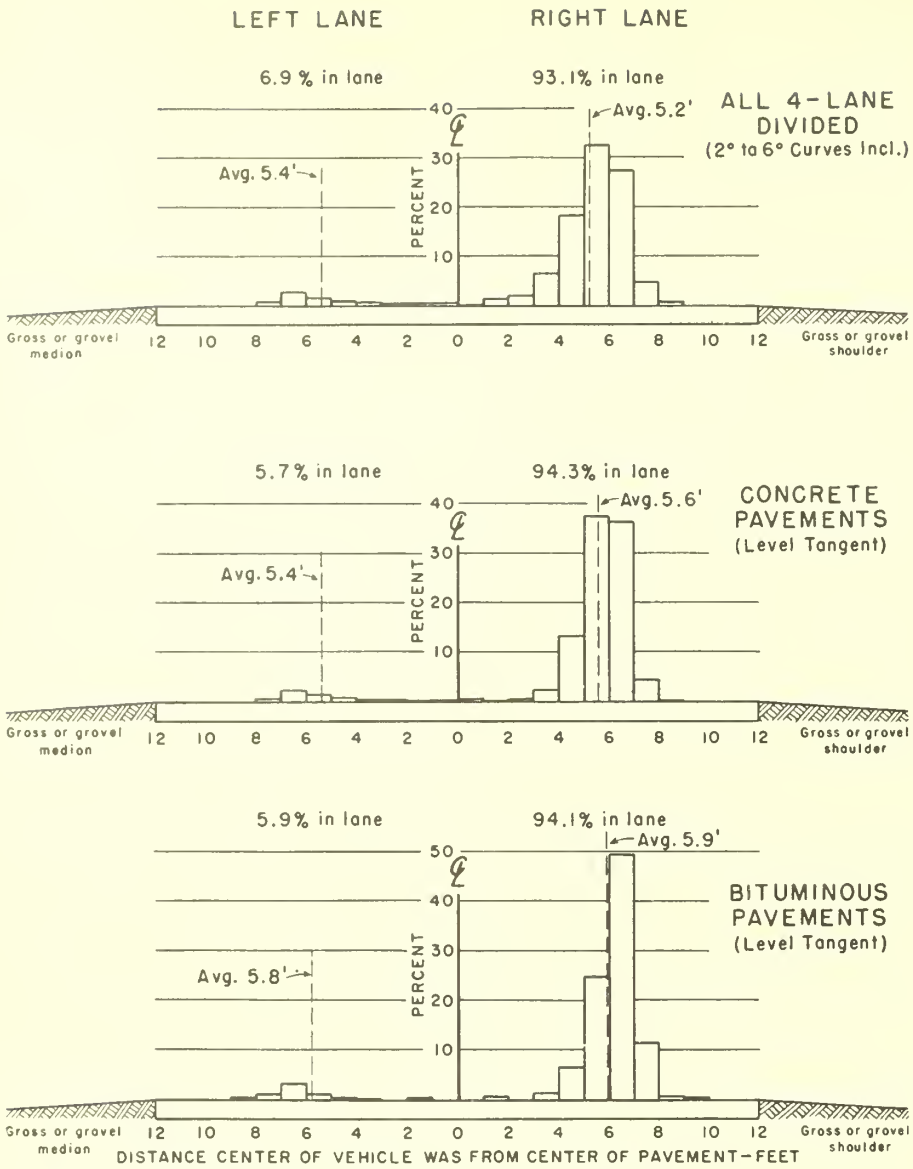

Figure 2.—Distribution of lateral positions of trucks on four-lane divided highways with grass or gravel shoulders and median.

shoulders and traffic lanes. On these sections, which did not have edge markings, nearly 70 percent of all trucks traveled partly on the shoulder. Of the trucks meeting other vehicles (not shown in table 4), nearly 80 percent encroached on the shoulder. Encroachment on the left traffic lane varied from 1.2 to 4.3 percent. The greatest encroachment on the left lane occurred on bituminous pavements with combination-type shoulders (part bituminous and part gravel), 8 to 10 feet in width, whereas the least encroachment was found on sections with bituminous-paved traffic lanes and shoulders.

Lateral Placements on Four-Lane Highways

Table 5 and figure 2 show truck placements in the left and right lanes of four-lane divided highways. Of a total of 39 locations studied, 27 locations were on level tangent sections paved with portland cement concrete. The placements in the right lane on level sections of four-lane divided highways were very similar to the placements on two-lane highways. On horizontal curves, the placements on the four-lane divided highways were closer to the center of the pavement than on the two-lane roads. Because of the low traffic volume, only about 7 percent of the trucks traveled in the left traffic lane of divided highways. On the level tangent sections, the lateral position of trucks in the left lane with respect to the center of the pavement was about the same as the lateral position in the right lane.

Figure 3 shows the relation between the hourly volumes and the percentages of vehicles traveling in the right lane on a four-lane divided highway. At a volume of 500 or less vehicles per hour, at least 90 percent of the commercial vehicles and 73 percent of the passenger cars used the right lane. The percentage of trucks using the right lane decreased to 75 percent with an increase in the volume to 2,000 vehicles per hour. Above this volume

Table 6.—Number of locations, size of sample, average speeds, and average placements of trucks included in studies on two-lane highways and four-lane divided highways

Surface types of pavements with grass or gravel shoulders	Two-lane highways				Four-lane divided highways						
					Number of locations	Left traffic lane			Right traffic lane		
	Number of locations	Size of sample	Average speed	Average placement [1]		Size of sample	Average speed	Average placement [1]	Size of sample	Average speed	Average placement [1]
LEVEL TANGENT SECTIONS											
			M. p. h.	*Ft.*			*M. p. h.*	*Ft.*		*M. p. h.*	*Ft.*
Portland cement concrete pavements	7	1,473	43.5	6.0	27	348	52.3	5.4	4,602	46.2	5.6
Bituminous pavements	25	4,206	46.8	5.8	3	49	47.4	5.8	585	44.1	5.9
LEVEL CURVES (2 to 6 degrees)											
Portland cement concrete pavements: [2]											
Curve to right	2	169	43.2	5.7	3	20	46.7	5.2	475	40.7	5.1
Curve to left		192	41.4	5.8	4	62	53.0	5.0	454	49.0	4.6
Bituminous pavements: [2]											
Curve to right	6	447	42.7	6.2	-------	-----	-----	-------	-----	-----	-------
Curve to left		447	42.0	5.3	2	16	43.6	5.6	189	42.0	5.0
SUMMARY											
All pavement sections	40	6,934	43.3	5.8	39	495	48.6	5.4	6,305	44.4	5.2

[1] Distance from center of truck to centerline of pavement. [2] For two-lane highways, "curve to the right" indicates an inside lane and "curve to the left" indicates an outside lane.

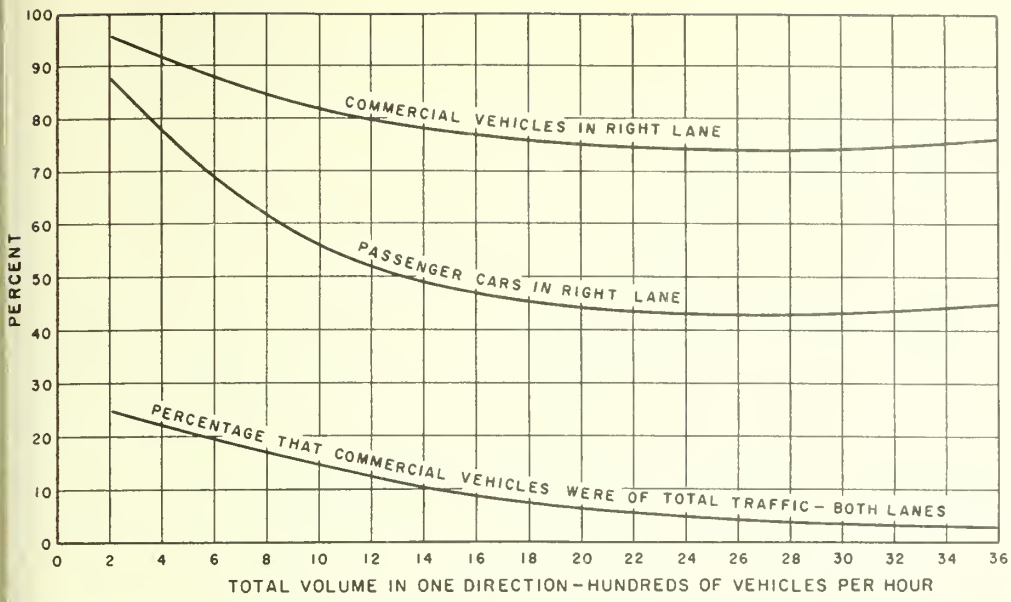
COMMERCIAL VEHICLES IN RIGHT LANE
PASSENGER CARS IN RIGHT LANE
PERCENTAGE THAT COMMERCIAL VEHICLES WERE OF TOTAL TRAFFIC - BOTH LANES
PERCENT
TOTAL VOLUME IN ONE DIRECTION - HUNDREDS OF VEHICLES PER HOUR

Figure 3.—Percentage of vehicles using right-hand lane of a four-lane divided highway and the percentage of commercial vehicles in the traffic stream during various hourly traffic volumes.

Table 7.—Distribution of placements of trucks on rural two-lane level tangent pavements of different widths and shoulder surface types

Distance from center of truck to centerline of pavement [1]	Distribution of placements on 10-foot traffic lanes with—		Distribution of placements on 11-foot traffic lanes with—		Distribution of placements on 12-foot traffic lanes with—	
	Contrasting bituminous shoulders	Grass or gravel shoulders	Contrasting bituminous shoulders	Grass or gravel shoulders	Contrasting bituminous shoulders	Grass or gravel shoulders
Feet	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
0.0–0.9	----------	0.2	0.2	0.2	0.2	0.1
1.0–1.9	0.5	.2	.2	.2	.1	.1
2.0–2.9	1.0	.8	.1	1.2	.2	.4
3.0–3.9	4.9	3.3	1.9	5.0	.7	.9
4.0–4.9	30.8	32.6	7.8	19.1	4.4	9.0
5.0–5.9	23.8	52.2	30.0	41.2	14.6	29.5
6.0–6.9	14.1	9.6	37.8	30.3	31.3	44.5
7.0–7.9	13.1	.4	10.8	2.8	18.3	13.3
8.0–8.9	7.9	.1	4.0	[2]	13.3	1.6
9.0–9.9	3.9	.5	2.1	----------	6.1	.2
10.0–10.9	----------	.1	2.1	----------	3.6	.1
11.0–11.9	----------	----------	1.2	----------	2.4	.2
12.0–12.9	----------	----------	1.6	----------	2.4	.1
13.0–13.9	----------	----------	.2	----------	2.0	----------
14.0–14.9	----------	----------	----------	----------	.1	----------
15.0–15.9	----------	----------	----------	----------	.2	----------
16.0–16.9	----------	----------	----------	----------	.1	----------
Total	100.0	100.0	100.0	100.0	100.0	100.0

[1] For distances less than 4 feet, trucks were straddling the centerline of the pavement; for distances greater than 6, 7, and 8 feet for 10-, 11-, and 12-foot traffic lanes, respectively, trucks were encroaching on the shoulder.
[2] Less than 0.05 percent.

Table 8.—Number of locations, size of sample, average speeds, and average placements of trucks on two-lane highways with 10-, 11-, and 12-foot traffic lanes

Lane widths and shoulder types	Number of locations	Size of sample	Average speed	Average placement [1]
			M. p. h.	*Feet*
10-foot traffic lanes with—				
Contrasting bituminous shoulders	2	140	42.6	5.7
Grass or gravel shoulders	8	716	42.6	5.0
11-foot traffic lanes with—				
Contrasting bituminous shoulders	6	1,142	44.4	6.3
Grass or gravel shoulders	10	1,525	40.5	5.3
12-foot traffic lanes with—				
Contrasting bituminous shoulders	15	2,926	48.4	7.1
Grass or gravel shoulders	32	5,679	45.2	5.9

[1] Distance from center of truck to centerline of pavement.

the percentage of trucks in the right lane remained constant at 75 percent. In other words, the left lane of travel was used by less than 25 percent of the trucks during the heaviest traffic volumes, and by less than 10 percent of the trucks during volumes below 500 vehicles per hour.

Trucks constituted a decreasing proportion of the total traffic as the volume increased. At volumes of 500 vehicles per hour for one direction of travel on a four-lane divided highway, 20 percent of the vehicles were trucks; at volumes of 3,500 vehicles per hour only 3 percent of the vehicles were trucks. The results shown in figure 3 are based on recent data recorded on a principal four-lane divided highway south of Washington, D. C.

Table 6 is a summary of the studies made on both two-lane highways and four-lane divided highways. Included are the number of locations studied, the size of samples, the average speeds of vehicles, and the average placements with respect to the centerline of the pavements.

In addition to placement data on pavements with 12-foot lanes, there is interest in data on pavements with lane widths other than 12 feet. Table 7 shows the distribution of placements of trucks on two-lane roads with 10-, 11-, and 12-foot lanes including both bituminous and concrete pavements. For each lane width the data are shown separately for sections with bituminous shoulders and grass or gravel shoulders.

As indicated in table 8 for traffic lanes of the same width, the placements on sections with bituminous shoulders are considerably farther from the centerline of the pavement than on sections with grass or gravel shoulders. The results of this study confirm the results of earlier studies [5] that bituminous-paved shoulders which appear distinctly different from the traffic lane increase the effective pavement width at least 2 feet, regardless of the lane width.

[5] See footnote 2, p. 71.

Highway Statistics, 1956

The Bureau's HIGHWAY STATISTICS, 1956, the twelfth of the bulletin series presenting annual statistical and analytical tables of general interest on the subjects of motor fuel, motor vehicles, highway-user taxation, financing of highways, and highway mileage is now available.

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VOL. 30, NO. 4

OCTOBER 1958

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U. S. DEPARTMENT
OF COMMERCE,
WASHINGTON

In this issue: Two studies of trip patterns in the Washington, D. C., area. (Shirley Memorial Highway and Glebe Road interchange, Arlington County, Va.)

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Evaluating Trip Forecasting Methods With an Electronic Computer

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported [1] by **GLENN E. BROKKE** and **WILLIAM L. MERTZ**, Highway Research Engineers

In forecasting trip distribution from the existing pattern, the average factor method, the Detroit method, and the Fratar method are equally accurate if each method is carried through a sufficient number of successive iterations. In all cases tested, maximum accuracy was reached in the second approximation of the Fratar method while four or more approximations were usually required with either of the other two methods.

The results of the test emphasized that the majority of the trips within a metropolitan area consist of a very large number of small-volume zone-to-zone movements, where the zones are of normal size. With sampling rates used, these individual small-volume movements are not accurately determined. The accumulation of the small-volume movements into volumes associated with ramps, streets, and expressways should result in acceptable accuracy as calculated by statistical formulas, but this will have to be definitely established by additional research.

The advantage of using an electronic computer on research projects of this type can hardly be overestimated, even though preparing the program is difficult and time consuming. In this series of tests, the speed and accuracy of the computer permitted the attainment of results in hours instead of years after the program had been completed, without a single error attributable to the computer.

STUDIES of trip origins and destinations of residents in the Washington, D. C., area were made in 1948 and 1955. In the earlier study a 5-percent sample was obtained by interviewing the residents of one of every 20 dwelling units. In the 1955 study, the sample rate was 1 in 30 in the District of Columbia and 1 in 10 elsewhere within the area.

These two surveys offered, for the first time, an opportunity to study the changes occurring in the pattern of trips over a period of several years in a metropolitan area, i. e., the differences in the numbers of trips between the same origins and destinations. They also provided data that could be used to evaluate methods of forecasting trip volumes.

Trip Forecasting Elements

Two basic elements are involved in the forecasting of trips. One is the increase in the number of trip origins and destinations in a particular part of the city such as a zone. For brevity the number of trip origins and destinations combined have been labeled trip-ends. For example, 2 trips originating in a zone and 3 trips destined to it would be counted as 5 trip-ends. Manifestly, if only the trips made wholly within an area are considered, the total number of trip-ends in the area is exactly twice the number of trips.

The ratio of future trip-ends expected in a particular zone to present trip-ends is called the growth factor for that zone. Much work has been and is being done in this field to determine the best method of arriving at the proper growth factor. Up to now, forecasts, as far as total trip-ends are concerned, are dependent to some extent on personal judgment. In order to eliminate this variable and isolate the elements being studied, the growth factors were calculated for each zone by taking the ratio of the reported trip-ends in each zone in 1955 to the reported trip-ends in each zone in 1948. Thus any variability in predicting growth factors will not affect this study of forecasting methods.

The other basic element involved in forecasting zone-to-zone movements is the application of the growth factors of the two terminal zones in predicting the number of future trips between them. Various mathematical formulas have been developed with that end in view. It is the purpose of this study to evaluate the accuracy of these methods and the formulas used therein. Certain other methods of trip forecasting, based on population distribution, trip-attraction distribution, and distance or travel time, directly predict the number of zone-to-zone trips; but as these methods are still in the process of development they will not be discussed here.

Characteristics of the Area

One problem that had to be resolved in beginning the test was that the area covered in the 1948 Washington survey was somewhat smaller than that of the 1955 survey, and the extent and identifying numbers of many of the zones had been changed. The first step was to reconcile these differences by rezoning the metropolitan area into 254 zones and to determine both the 1948 and 1955 volumes of trips into and out of these zones.

The 254 zones covered an area which contained 96 percent of the population that lived within the 1948 cordon and 93 percent of that living within the 1955 cordon. Most of the external cordon stations in 1955 were placed at different locations from those in 1948; and therefore the trips crossing the cordon, called "external" trips, are omitted from the study, the two surveys not being comparable for this class of trip.

Within the 254 zones the population increased 38 percent during the 7-year interval, while the number of trips by persons [2] increased 42 percent. This represents a small increase in trips per person from 1.95 to 2.00. During the same interval the number of passenger cars owned by residents almost doubled, increasing 96 percent, while the number of trips made by these passenger cars went up 89 percent. This represents a small decrease in the number of trips per passenger car from 3.15 to 3.05. These comparisons seem to indicate that the number of automobile trips increases roughly in proportion to the increase in the number of automobiles, and total trips by persons increase about as population does in the Washington area.

The number of trips by various vehicle types and modes of transportation, together with population and passenger-car ownership, are shown in figure 1. The growth factors resulting from the changes between 1948 and 1955 are shown graphically in figure 2.

The increase of 89 percent in the number of passenger-car trips during the 7-year interval is rather high. It represents an average increase of almost 13 percent annually on a straight-line basis or about 9.5 percent if compounded annually.

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

[2] In this article "trips by persons" include trips by drivers of automobiles, taxis, and trucks and by passengers in automobiles, taxis, and mass-transit vehicles. Pedestrian trips and the small number of trips by passengers in trucks are not included.

This high rate of increase in the Washington area, however, has the advantage of providing growth factors that are somewhat similar to the growth factors that have been forecast for about 25 years in some of the larger cities. For example, the growth factors for total vehicle trips as measured in Washington and those predicted for Detroit and Cleveland are shown in table 1.

Thus while the test of the forecasting methods is confined to the growth of Washington from 1948 to 1955, the actual growth factors are not entirely dissimilar to forecasts for Detroit and Cleveland although the latter two are for longer periods of time.

In this study, passenger-car and taxi trips were combined into one category of passenger-vehicle trips. The overall growth factor for these trips was 1.67. As for individual zones, about 6 percent had fewer trip-ends in 1955 than in 1948, and 50 percent had a growth factor smaller than 1.55. A more detailed distribution of the individual zone growth factors for passenger-vehicle trips is given in table 1.

Methods of Forecasting Trips

The 1948 zone-to-zone trips were expanded to 1955 by various formulas and the predicted values were compared with those obtained in the 1955 sample. The actual methods are subsequently described as the uniform factor method, the average factor method, the Detroit method, and the Fratar method. Symbols used in mathematical formulas are as follows:

T_{ij} = Observed 1955 trips between zone i and zone j.

T_{ij}' = Calculated 1955 trips between zone i and zone j.

T_{i-j}' = Calculated 1955 trips from zone i to zone j.

T_{j-i}' = Calculated 1955 trips from zone j to zone i.

t_{ij} = Observed 1948 trips between zone i and zone j.

T_i = Summation of observed 1955 trip-ends in zone i.

t_i = Summation of observed 1948 trip-ends in zone i.

F_i = Growth factor for zone $i = T_i/t_i$.

T = Summation of 1955 trip-ends in entire area.

t = Summation of 1948 trip-ends in entire area.

F = Growth factor for entire area $= T/t$.

t_{ix} = 1948 trips between zone i and each of all other zones designated as zone x.

F_x = Growth factor for zone x.

Uniform Factor Method

The most simple method of expanding trips is to compute a single factor for the entire area and use it to multiply all zone-to-zone trips. This particular method is seldom used now, but because of its wide use in the past it was evaluated. Mathematically the expansion formula is $T_{ij}' = t_{ij} \times F$. There is no possibility of successive approximations with this method, such as those used in the methods subsequently described.

Average Factor Method

In the average factor method each of the 1948 zone-to-zone movements is multiplied by the average of the growth factors for the two zones involved, $T_{ij}' = t_{ij}(F_i + F_j)/2$.

After the trips from one zone, i, to all other zones have been computed by this method, the sum of all trip-ends in that zone, as determined from this calculation, T_i', will probably not equal the actual 1955 trip-ends in that zone, T_i. This discrepancy can be eliminated by a series of iterations producing successively closer approximations, as follows:

Let F_i' equal the factor needed to bring the calculated number of trip-ends T_i' to actual number T_i or $F_i' = T_i/T_i'$, and similarly $F_j' = T_j/T_j'$. Then for the second approximation, $T_{ij}'' = T_{ij}'(F_i' + F_j')/2$, and similarly for a third approximation, $T_{ij}''' = T_{ij}''(F_i'' + F_j'')/2$. The process can be repeated until the F factors for a new iteration equal the limiting value of 1.00.

One of the inherent disadvantages of the average factor method is that the calculated trips into zones with higher-than-average growth factors generally total less than the predicted number of trips Conversely the calculated trips into zones with lower-than-average growth factors total more than the predicted number of trips. This systematic bias of the predicted values could result in an inordinate number of approximations and might affect the accuracy of the method.

Detroit Method

A method to alleviate this difficulty was developed by Dr. Carroll's staff for the Detroit study.[3] In this method they assumed that the trips from zone i will increase as predicted by F_i and will be attracted to zone j in the proportion F_j/F. The predicted trips from zone i to zone j can then be calculated from the equation $T_{i-j}' = t_{i-j}(F_i \times F_j)/F$. Similarly the trips from zone j can be considered as increasing as predicted by F_j and will be attracted to zone i in the proportion F_i/F.

The predicted trips from zone j to zone i can then be calculated from the expression $T_{j-i} = t_{j-i}(F_j \times F_i)/F$. Therefore the number of trips between zone i and zone j is equal to the sum of the trips from i to j and from j to i or $T_{ij}' = T_{i-j}' + T_{j-i}'$.

Hence:

$$T_{ij}' = t_{i-j}\frac{(F_i \times F_j)}{F} + t_{j-i}\frac{(F_j \times F_i)}{F}$$

$$= (t_{i-j} + t_{j-i})\frac{(F_i \times F_j)}{F}$$

$$= t_{ij}\frac{(F_i \times F_j)}{F}$$

[3] *Report on the Detroit Metropolitan Area Traffic Study, Part II—Future Traffic and a Long Range Expressway Plan, March 1956.* J. D. Carroll, Jr., Study Director.

As in the case of the average factor method the calculated trip-ends in a particular zone will probably not equal the predicted trip-ends in that zone. Therefore new F factors can be determined as follows: $F_i' = T_i/T_i'$ and $F_j' = T_j/T_j'$. A second approximation can be calculated from the expression

$$T_{ij}'' = T_{ij}'(F_i' \times F_j')/F'.$$

This same procedure can be used to calculate a third and subsequent approximations until the new F factors equal the limiting value of 1.00.

Fratar Method

The first method in which the iterative process was used in predicting trips was developed by Thomas J. Fratar in connection with the forecast for Cleveland, Ohio. Fratar considers that the distribution of the trips from any zone i is proportional to the present movements out of zone i modified by the growth factor of the zone to which these trips are attracted. The volume of the trips, however, is determined by the expansion factor of zone i.

If the trips between zones i and j, as calculated by considering all trips from zone i, are represented by the symbol $T_{ij(i)}'$ and those as calculated by considering all of the trips from zone j by the symbol $T_{ij(j)}'$, then

$$T_{ij(i)}' = t_{ij} \times F_j \times \frac{\Sigma t_{ix} \times F_i}{\Sigma t_{ix} \times F_x} \text{ ----------(1)}$$

Noting that $\Sigma t_{ix} \times F_i$ can also be written as $F_i \times \Sigma t_{ix}$, then equation 1 can be written as

$$T_{ij(i)}' = t_{ij} \times F_j \times F_i \times \frac{\Sigma t_{ix}}{\Sigma t_{ix} \times F_x} \text{ -------(2)}$$

The last term in equation 2 represents basically the reciprocal of the average attracting pull of all other zones on i. It is labeled the "location" or L factor since it is somewhat dependent on the location of the zone with respect to all other zones. Since by definition $\Sigma t_{ix}/\Sigma(t_{ix} \times F_x) = L_i$, equation 2 can be rewritten as

$$T_{ij(i)}' = t_{ij} \times F_j \times F_i \times L_i \text{ ----------(3)}$$

Then for all trips from zone j, it can similarly be shown that

$$T_{ij(j)}' = t_{ij} \times F_i \times F_j \times L_j \text{ ----------(4)}$$

Thus the trips between zone i and zone j have been computed twice—once for all trips out of zone i and once for all trips out of zone j. The most probable value is an average of the two computations or

$$T_{ij}' = \frac{T_{ij(i)}' + T_{ij(j)}'}{2} \text{ ----------(5)}$$

Substituting the identities from equations 3 and 4 in equation 5 and factoring out the common terms, the final equation is developed as follows:

$$T_{ij}' = t_{ij} \times F_i \times F_j \times \frac{L_i + L_j}{2} \text{ ----------(6)}$$

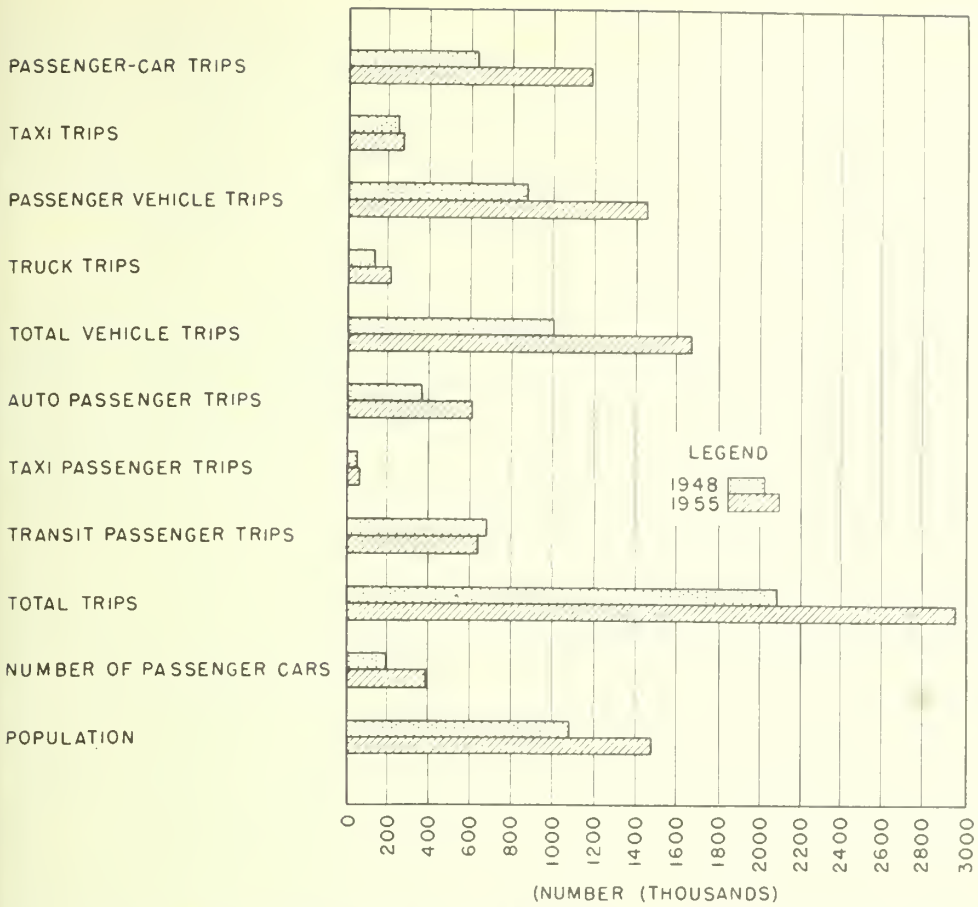

Figure 1.—Number of trips within the Washington, D. C., area by various modes of transportation, together with passenger-car ownership and population data, 1948 and 1955.

After all the zone-to-zone trips have been computed by this formula, the calculated trip-ends in a particular zone will probably not agree with the predicted trip-ends in that zone. Therefore new factors can be calculated as follows: $F_i' = T_i/T_i'$, $F_j' = T_j/T_j'$, $L_i' = \Sigma T_{ix}'/\Sigma(T_{ix}' \times F_x')$, and $L_j' = \Sigma T_{jx}'/\Sigma(T_{jx}' \times F_x')$. A second approximation can then be calculated as $T_{ij}'' = T_{ij}' \times F_i' \times F_j' \times (L_i' + L_j')/2$. The same procedure can be used for subsequent approximations until the new F factors equal the limiting value of 1.00.

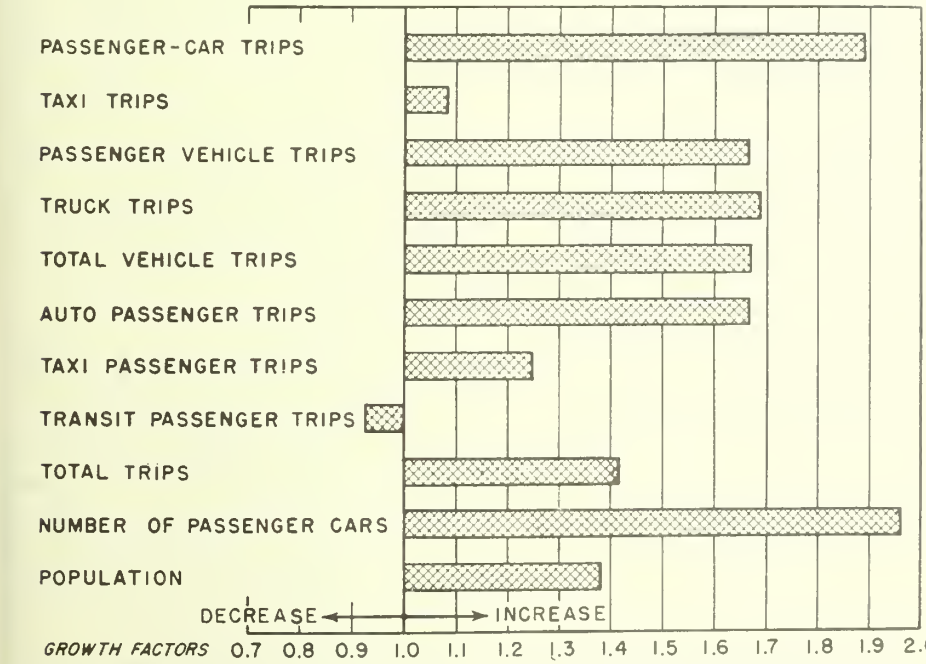

Figure 2.—Changes in the growth factors of trips by various modes of transportation, passenger-car ownership, and population, 1948–55.

Problem of Evaluation

With the Washington area divided into 254 zones, the number of possible zone-to-zone movements [4] is $[N(N+1)]/2 = 32{,}385$. It was expected that some of the zone-to-zone movements would be zero in 1948 and 1955 and would not need to be computed. However, it was estimated conservatively that perhaps 30,000 of the 1948 zone-to-zone movements would require expansion to 1955.

To determine whether the accuracy of the prediction was influenced by vehicle type or mode of transportation, the trips were separated by mode of travel into six categories: Passenger-vehicle trips, truck trips, total vehicle trips, mass-transit passenger trips, automobile passenger trips, and total trips by persons. Each group was expanded separately.

Expanding each of the 30,000 zone-to-zone trips made in 1948 would be almost meaningless unless some method of summarizing the comparison to the 1955 survey movements had been determined. The most obvious answer to this problem was to subtract the computed number of trips from the reported number of 1955 trips, square the differences, and accumulate the results. The sum of the differences squared could then be used to calculate the root-mean-square error of the number of trips as expanded from the 1948 data.

This summary, however, had a serious disadvantage in that the actual volume of zone-to-zone trips varies from zero to several thousand. A root-mean-square error could be inordinately affected by the relatively few large movements. Similarly, if the differences were converted to a percentage of the 1955 actual movement and the root-mean-square of the percentage computed, the result could be as greatly affected by the small movements that probably lack sufficient stability to provide meaningful information. It was therefore decided to stratify the 1948 movements by volume classes: by volumes of tens to 100, by volumes of hundreds to 1,000, and all volumes over 1,000. The numerical root-mean-square error was then computed for each volume class, and the percentage error for the class was obtained from the ratio of the numerical root-mean-square to the average 1955 volume. The proportion of all 1955 volumes in each volume class was then determined and each of the percentage errors was weighted by the proper proportion to obtain the overall percentage error.

The overall percentage error was regarded as the proper measure to evaluate the various predictive formulas. In addition, the accuracy of larger movements could be measured by an extrapolative process in which the number of average zone-to-zone movements required to make up a larger volume was determined and the basic error was divided by the square root of the number of movements required. In addition to this computation, it was desirable to know the root-mean-square error for each of the zones so that the error could be related

[4] Zone-to-zone movements as used in this article also include intrazone movements.

Table 1.—Comparison of growth factors for total vehicle trips in three major cities, and passenger-vehicle trips for Washington, D. C., only

City	Period of study or forecast	Overall growth in trip volumes	Percentage of zones with growth factors of—						
			Less than 1.00	1.00–1.49	1.50–1.99	2.00–2.99	3.00–4.99	5.00–9.99	Over 10.00
TOTAL VEHICLE TRIPS									
Washington, D. C.	1948–55	1.66	5	38	25	18	9	3	2
Detroit, Mich.	1953–80	1.67	2	64	9	9	8	6	2
Cleveland, Ohio	1952–75	1.79	1	54	17	8	15	5	0
PASSENGER-VEHICLE TRIPS									
Washington, D. C.	1948–55	1.67	6	40	23	19	7	2	3

to the growth factor of the individual zones. Therefore, the difference between the expanded 1948 and 1955 movements squared was accumulated for each of the zones. It was considered likely that from this computation any inordinate error found in a particular zone could be recognized.

As has been previously explained, it is possible to carry the average factor method, the Detroit method, and the Fratar method through a number of iterations to produce successively closer approximations. To be reasonably certain that this process was continued a sufficient number of times, it was decided to calculate 10 successive approximations by each method.

Need for an Electronic Computer

It has been estimated that roughly 25 million computations would be required for this test. On the very optimistic basis that one computation can be completed in 10 seconds, using ordinary desk calculators, the project would require some 30 man-years for completion. Clearly this is not feasible. However, with electronic computers, the time required can be greatly reduced.

The three methods that use iterations are similar in that the input for the first calculation is made up of the original data, the output of this calculation and each successive iteration becomes the input of the next iteration, and so on until 10 calculations have been made if necessary for satisfactory closure. In addition each iteration of the Fratar method requires two passes of the input—one to determine the L factor and one to make the required expansion. Thus, the 30,000 zone-to-zone movements have to be processed 42 times, including the one pass of the data required to obtain the original growth factors.

In deciding upon the particular type of computer to be used, the first problem was to decide whether to use one with card input and output, or one with tape. The card type would require about 200 hours of computer time provided sufficient memory were available. With tape, only about 10 hours of computer time or less would be required, again assuming sufficient memory; obviously the tape-using type was preferable. In the actual test, 30 hours of computer time were used principally due to additional tests on larger zone groupings.

Table 2.—Root-mean-square error in the number of zone-to-zone trips forecast for 1955 from 1948 data, compared with 1955 survey results

Approximation number	Numerical RMS error [1]				Percentage RMS error [2]			
	Uniform factor method	Average factor method	Detroit method	Fratar method	Uniform factor method	Average factor method	Detroit method	Fratar method
PASSENGER CARS								
1	165	133	234	140	151	136	192	140
2		132	129	131		136	133	134
3		133	148	132		136	143	134
4		134	129	132		137	133	135
5		134	136	132		137	137	135
6		135	131	132		138	134	135
7		135	133	132		138	135	135
TRUCKS								
1	78	57	59	55	163	160	172	162
2		55	58	55		160	161	161
3		55	55	55		161	163	161
4		55	56	55		162	161	161
5		55	55	55		162	162	161
6		55	55	55		162	161	161
7		56	55	55		162	162	161
TOTAL VEHICLES								
1	174	137	229	138	141	124	175	125
2		133	131	130		122	120	120
3		133	144	131		121	128	121
4		133	129	131		122	119	121
5		133	134	131		122	122	121
6		133	130	131		122	120	121
7		133	132	131		122	121	121

[1] Calculated on the basis of number of zone-to-zone movements that had more than zero trips in either 1948 or 1955. Average zone-to-zone volumes were as follows: passenger cars, 84; trucks, 28; and total vehicles, 90.

[2] Calculated by determining the error in the various volume groups and weighting the error in each group in proportion to the percentage of all trips in that group.

Computer Characteristics

Computer problems in general fall into two categories: data-processing problems and computation problems. For instance, a problem of testing forecasting methods, as discussed herein, requires a great deal of input and output but rather simple internal operations and is properly classified as a data-processing problem. On the other hand, computing such things as log tables or trigonometric functions requires much computation but very little input and output.

The problem then was to select a machine designed to process data with good "read and write" characteristics and a large memory. Arrangements were made for part-time use of an IBM 705 machine which met these requirements. The machine had a core memory of 40,000 characters. The memory capacity of computers is sometimes reported in "characters" and sometimes in "words." In computer terminology, a character may be a digit, a letter or a symbol, while a word consists of a group of characters. In some computers the word is of a constant length, and in other computers the word length may be varied at the option of the programer. The computer used was a variable-word-length machine and the core-memory capacity is therefore given in characters rather than words.

The machine was equipped with two tape-record coordinators, more commonly referred to as buffers. The purpose of the buffers is to shorten the reading and writing time. Each buffer has a core storage capacity of 1,024 characters. The buffers are loaded from the tape units, and when the computer requires more data it obtains it at electronic speed from the buffer. As soon as the buffer is called upon for data, the tape unit feeding it begins to accelerate, so that by the time the buffer is empty the tape unit has begun to refill it with the next record to be processed. The same process works essentially in reverse for output. Thus the machine can go on with other work while records are being fed into and out of the buffers.

The 1,024-character capacity of the buffers also allows a number of card records to be grouped so that when the buffer calls on the tape units for data, one tape record can receive information from a number of cards without exceeding the buffer capacity. The program for this study was designed to have the machine read or write the equivalent of 24 cards of information per reading or writing cycle. As it turned out in production, the computer took a longer time to process the data on the 24 cards than was required to fill or empty the buffers so that the machine never had to wait for data, and all reading or writing time was essentially "free."

Table 3.—Variation in number of trips and reduction in percentage error with increasing zone size

Number of areas	Minimum number of trip-ends per area	Average number of trip-ends per area	Number of area-to-area possibilities	Average number of area-to-area trips	Minimum percentage of RMS error
254 zones	193	6,900	32,385	27	133
122-zone groups	10,000	14,400	7,503	116	70
66-zone groups	20,000	26,600	2,211	394	41
49-zone groups	30,000	35,800	1,225	711	34
7-zone groups	214,305	250,000	28	31,092	14
2-zone groups	731,000	870,000	3	290,192	11

The IBM 705 is a decimal machine, meaning that the entire content of memory is in condition always to be printed out as alpha-numeric information directly without conversion from binary to decimal. The machine performs internal operations at an average rate of 8,300 per second.

Preparation of Program

Since this was the first large-scale electronic computer project undertaken by the Bureau of Public Roads, the services of the National Bureau of Standards Computer Laboratory were retained to aid in the selection of the machine and in the preparation of the program. Their wide experience in this field has proved invaluable in the completion of this work.

In programing the problem the first difficulty was one of memory space, in spite of the large amount available. It was necessary to overlap the program and resort to external tape storage. Even so it was necessary to split the problem into two parts. All of the vehicle types—passenger cars, trucks, and total vehicles—were handled in the first run. The second run processed the trips by persons: automobile passengers, mass-transit passengers, and total persons. The same basic program was used for both runs, however.

It was necessary to prepare a preliminary program in order to group the single card records into groups of 24, which is the maximum number of cards within buffer capacity, and to separate them into vehicle or person categories. Included in this preliminary program were an editing routine, a volume-classification routine, and a check-sum routine. The editing routine rejected any cards with alphabetic information, i. e., over-punches, inconsistent zone numbers, etc. The volume-classification routine classified the 1948 volumes of trips into 20 volume classes. The check-sum routine summed all the volumes by modes and zones and compared the totals with a 254-card summary deck prepared independently. The preliminary program also permitted the preparation of the main program while the input data were being compiled. Any changes in the arrangement of the data taken from the cards could be taken care of in the first program without affecting the main program.

Table 4.—Root-mean-square error of the 1955 passenger-car trips forecast for the 254 zones and for different zone groupings

Approximation number	254 zones, error in trips		122-zone groups, error in trips		66-zone groups, error in trips		49-zone groups, error in trips		7-zone groups, error in trips	
	Number	Percent [1]	Number	Percent [1]	Number	Percent [1]	Number	Percent [1]	Number	Percent [1]
UNIFORM FACTOR METHOD										
-----	165	151	415	98	499	57	747	49	19,641	38
AVERAGE FACTOR METHOD										
1	133	136	244	77	458	49	655	42	10,180	20
2	132	136	204	72	364	44	542	37	8,300	16
3	133	136	196	71	344	43	517	36	7,810	15
4	134	137	194	71	338	42	509	35	7,690	16
5	134	137	193	71	337	42	507	35	7,540	15
6	135	138	193	71	336	42	506	35	[2]	------
7	135	138	193	71	336	42	506	35	--------	------
DETROIT METHOD										
1	234	192	388	89	720	64	871	51	10,300	20
2	129	133	228	74	396	46	543	37	8,960	17
3	148	143	299	74	344	42	504	35	7,700	15
4	129	133	194	70	337	42	484	34	7,510	15
5	136	137	196	70	320	41	478	34	7,500	15
6	131	134	189	70	326	41	480	34	7,570	15
7	133	135	190	70	319	41	476	34	7,480	14
FRATAR METHOD										
1	140	140	205	71	339	42	498	35	7,360	14
2	131	134	188	70	322	41	480	34	7,460	14
3	132	134	188	70	322	41	478	34	[2]	-- ---
4	132	135	188	70	321	41	479	34	--------	------
5	132	135	188	70	322	41	478	34	--------	------

[1] A weighted percentage error was obtained by determining the percentage error in each volume class and weighting this error by the proportion of trips in that volume class. Not applicable for the 7-zone group because all volumes were in the largest volume class.

[2] No further iterations required because new F factor for all zones was 1.00.

The program was designed to have all the final output written on tape for subsequent printing. It was found that the machine would be slowed down a great deal if a printer were connected "on line." As originally designed, there was one line of printed information for each of 20 volume classes times 6 modes of travel, one line for each of 254 zones times 6 modes, and one line for each overall citywide volume for each of the 6 modes. For all of the methods tested, with their iterations, this amounted to 52,800 printed lines, or almost 1,760 pages—a real data-processing problem.

Test Results

The initial run of the computer was made for vehicle trips by passenger cars, trucks, and total vehicles. The 1948 zone-to-zone movements were projected to 1955 by a uniform factor, by the average factor method, by the Detroit method, and by the Fratar method. These projected or forecasted results were compared with the measured 1955 volumes and the differences were squared, accumulated, and used to compute a root-mean-square error for the average movement, for the various volume classes, and for the individual zones.

These results are shown in table 2 for the three types of vehicle trips. Insofar as number of trips is concerned, the errors are not large, considering that the sample was as small as 1 in 30 for an important part of the data, and in no case larger than 1 in 10. On a percentage basis, however, the errors are very large.

When the results of the first computer run became available, the question immediately arose whether the errors were primarily attributable to the forecasting methods being tested or to the preponderance of low-volume zone-to-zone movements, which are known to lack accuracy or stability at the sample rates used.

This problem was attacked by two methods: One was by a systematic enlargement of the zones to increase the volume of the zone-to-zone movements and then testing these larger volumes through the computer program; the other method was to determine the percentage distribution of the zone-to-zone trip volumes within the city, and by statistical techniques determine the accuracy that might be expected in the original trip expansion. If the 1948 zone-to-zone trip volumes as expanded from the sample were unreliable, the error would be carried on into the forecast data, and if the 1955 expanded volumes were also unreliable, the result could compound the effect of the errors due to sample variability in comparing the forecasts with the 1955 data.

Enlarging Zones

Inasmuch as zone boundaries are chosen from land-use and geographic features, the number of trip-ends in each zone is not uniform. In this study the variability was intensified by the fact that the area had to

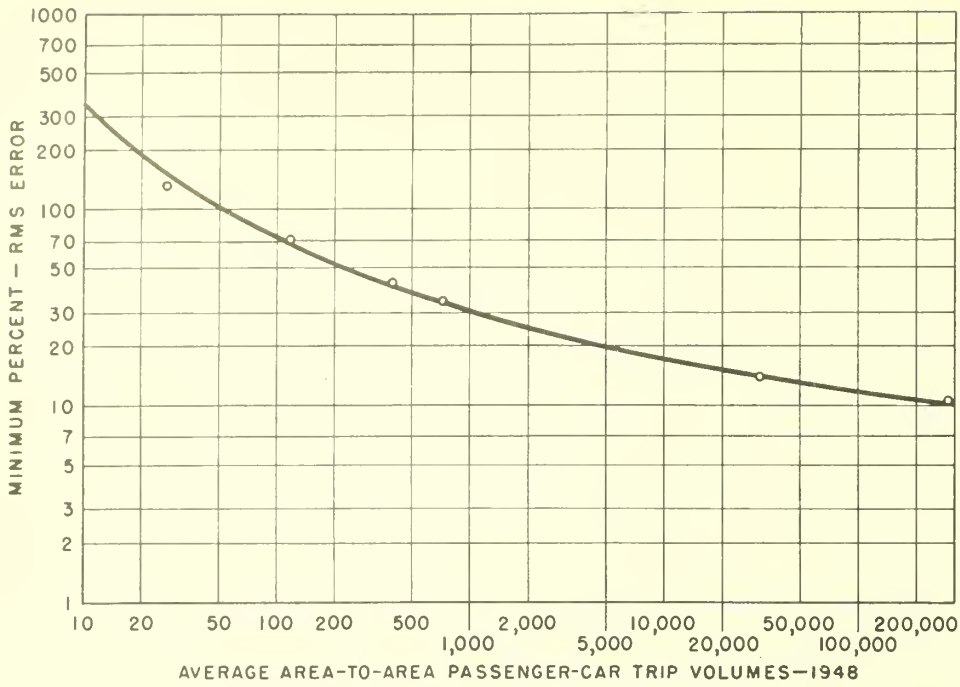

Figure 3.—Relation of average 1948 area-to-area passenger-car trip volumes and minimum percentage root-mean-square error.

be rezoned so that it would be identical in both years, and the trip-ends in the individual zones vary over wide limits. For example, the number of 1948 passenger-vehicle trip-ends averaged 6,900 per zone but varied from as few as 193 to as many as 59,870. As the initial step, therefore, adjacent zones were combined until each zone group had a minimum of 10,000 passenger-vehicle trip-ends in 1948. To minimize the effect of sample variability on the errors, this procedure was repeated to accumulate a minimum of 20,000 trip-ends per zone group, and again to accumulate a minimum of 30,000 trip-ends per group, then to divide the entire area into 7 groups, and finally into 2 groups.

The number of zones in these successive groupings, the number of 1948 passenger-car trip-ends in the average zone, and the average number of area-to-area trips are given in table 3.

Test Results of Enlarged Zones

The results of tests of each forecasting procedure for the various zone groupings are shown in table 4. As can be seen from this table, the average factor method, the Detroit method, and the Fratar method each reach essentially the same minimum error although the Fratar method reaches this minimum in the second approximation, whereas more iterations are generally required for the other methods.

The minimum percentage error, by any of the methods tested after any number of iterations for the various zone groups, and the average number of area-to-area passenger-car trips are shown in table 3.

In the case of the 2-zone group division, a second test was made by dividing the area with a line roughly at right angles to the first. The minimum percentage error for this second grouping was the same as that for the first, to the nearest percent (11 percent).

The relation between average area-to-area trip volume and the minimum percentage error is shown in figure 3. Since the minimum error for the three iterative methods is about the same, the chart can be considered as applicable to any one of them. Figure 3 is difficult to interpret because part of the error is due to sample variability and part is due to the projection method being tested. Differences in sampling rates introduce a further complication. In the 1948 survey the sampling ratio was 1:20 throughout the area, while in the 1955 survey it was 1:30 for the District of Columbia and 1:10 for the Maryland and Virginia suburbs. However, the curve should give some indication of the error to be expected in using any of the three iterative methods where the sampling rate is about the same as the average for the two Washington surveys; that is, about 5 percent.

The shape of the curve suggests that it will level off at about 10 percent. In other words an error of about 10 percent seems to be inherent in the methods tested, regardless of size of sample or areas.

Table 5.—Factor residual error for various iterations of the average factor, the Detroit, and the Fratar methods

Approximation number	Percentage of zones with a factor residual error of—						
	0.00	Less than 0.01	Less than 0.02	Less than 0.03	Less than 0.05	Less than 0.10	0.10 and over
Average Factor Method							
1	1	8	11	17	26	47	53
2	6	15	24	35	50	75	25
3	10	30	44	55	77	93	7
4	19	47	71	84	98	99	1
5	33	70	88	93	98	99	1
6	49	84	92	94	98	100	0
7	64	92	97	99	100	100	0
Detroit Method							
1	2	4	9	14	23	44	56
2	1	3	11	15	22	57	43
3	4	13	25	37	63	95	5
4	5	18	38	58	95	98	2
5	10	36	68	93	98	99	1
6	16	62	95	97	98	100	0
7	28	85	98	99	100	100	0
8	37	96	99	100	100	100	0
Fratar Method							
1	6	20	33	54	68	79	21
2	60	97	100	100	100	100	0
3	98	100	100	100	100	100	0
4	99	100	100	100	100	100	0

Rate of closure

A measure of the efficiency of the various forecasting methods is the rapidity with which the individual zone growth factors converge toward the limiting F factor of 1.00 in successive iterations. The difference between the computed F factor at the end of an iteration and 1.00 is the factor residual error that remains in the individual zones.

The factor residual error for the 254 zones is shown in table 5 for the various iterations of the three methods. The first column indicates the approximation number; the second column shows the percentage of the zones that have no residual error (new F factor=1.00) at the end of the approximation shown in the first column; the third column indicates the percentage of zones with a residual error less than 0.01 (new F factor between 0.99 and 1.01); the next four columns similarly show the percentage of zones with residual errors less than 0.02, 0.03, 0.05, and 0.10; and the last column shows the percentage of zones with residual errors greater than 0.10 (new F factor less than 0.90 or more than 1.10).

It can be seen from this table that the Fratar method is extremely efficient in its rate of closure. Since the F factor must be obtained for each new iteration and since

Table 6.—Average residual error related to root-mean-square error for the average factor, Detroit, and Fratar methods

Approximation number	Average factor method		Detroit method		Fratar method	
	Average residual error	RMS error	Average residual error	RMS error	Average residual error	RMS error
		Pct.		*Pct.*		*Pct.*
1	0.084	42	0.123	51	0.035	35
2	.034	37	.070	37	.003	34
3	.014	36	.032	35	.001	34
4	.007	35	.022	34	(1)	34
5	.003	35	.014	34	(1)	34

[1] Less than 0.001.

these new F factors may be easily summarized, it is suggested that they be used to indicate the desirability for additional iterations.

Number of iterations required

From the tests that have been run, the minimum root-mean-square error has always been reached in the second approximation by the Fratar method. By the other methods, however, this minimum error may not be reached until the fourth or fifth approximation. There is also a possibility that an unusual set of growth factors will develop that will not close as rapidly as those occurring in the test data.

Considering the division of the Washington, D. C., area into 49-zone groups with a minimum of 30,000 passenger-car trip-ends per group, the factor residual error was accumulated for all groups at the end of each iteration. The accumulated residual error was then divided by the number of groups to obtain the average residual error per group. This average residual error was then related to the RMS error already computed for each approximation. Table 6 indicates the results.

To be reasonably certain that greater accuracy cannot be obtained with additional iterations, it is suggested that iterations be continued until the average residual error per zone is less than 0.01.

The computer time required for each method, however, is not uniform but is approximately related to the complexity of the method. During the test, the computer time as recorded for each iteration of each method and adjusted proportionately to a common base of 10,000 zone-to-zone movements was as follows:

	Minutes
Average factor method	6
Detroit method	9½
Fratar method	12

The tabulation shows that the computer time required for the average factor method is half that required for the Fratar method. Included is the computer time needed to develop and store the various statistical measures. In an ordinary forecasting procedure these measures would not be required and the time periods would be reduced by a constant but indeterminate amount. The average factor method should therefore require something less than half the time per iteration required by the Fratar method.

Since the RMS error for the average factor method at the end of four approximations is about equal to the RMS error for the Fratar method at the end of two approximations, the overall computer time required for equal RMS accuracy is about the same. However, the rate of closure of the Fratar method is more than twice as rapid as the average factor method, and it would therefore appear to be the preferred method.

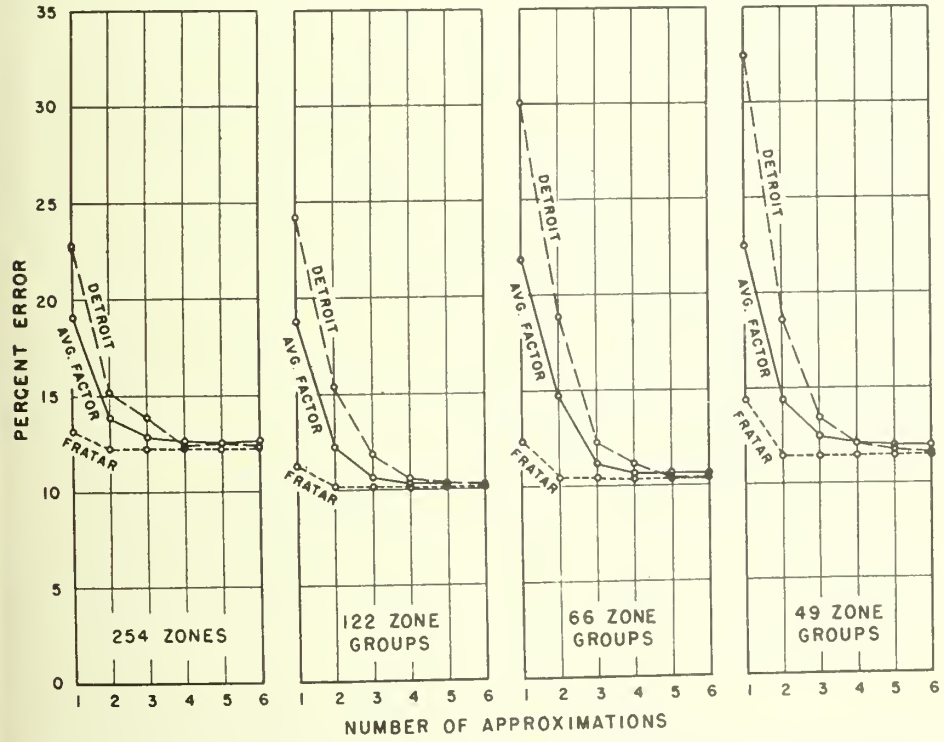

Figure 4.—Percentage error to be expected for zones grouped to provide volumes of 10,000 or more trips, based on the Detroit, average factor, and Fratar methods.

Percentage RMS Error For Accumulated Volumes

The data presented thus far have to do only with the error for the area-to-area volumes. As has been shown, the average volume between zones of the size ordinarily used is relatively small and the percentage root-mean-square error is, roughly speaking, correspondingly large.

In actual practice, the individual zone-to-zone volumes are assigned to the highway network and, therefore, each portion of the highway network represents an accumulation of zone-to-zone volumes. The volumes assigned to the highway network are our primary concern. The errors to be expected in such accumulated volumes can only be determined from actual tests and these have not yet been made. However, some indication of the magnitude of the errors to be expected can be obtained from purely theoretical considerations.

From a statistical standpoint, if the percentage error of an average zone-to-zone volume is X, the percentage error of a group of average zone-to-zone volumes is $X/\sqrt{N}$, where N is the number of individual zone-to-zone movements in the group. By dividing, 10,000 by the average zone-to-zone volume for each of the zone groupings, the number of zone-to-zone movements, N, required to accumulate to a volume of 10,000 can be determined, and the percentage RMS error of the group can therefore be calculated. The relation holds true only if the mean error of the group is zero. If the movements, however, are heavily weighted by trips from an individual zone, as they would be in the case of ramp volumes, the factor residual error, as previously explained, may be appreciable in the early iterations.

For example, if the trips on a ramp are essentially from two zones and these two zones have an average F factor for the next iteration of 1.30, the summation of trips into and out of the zones at the end of the present iteration is too low. The total number of trips for a group of zone-to-zone movements from these zones, therefore, will have a tendency to approach a volume which should be increased by 30 percent. To take this error into account, the root-mean-square of the residual errors was determined for each iteration of each method. The number of zones required to provide a volume of 10,000 was determined, and the root-mean-square residual error was added to the RMS error for individual zone-to-zone movements by taking

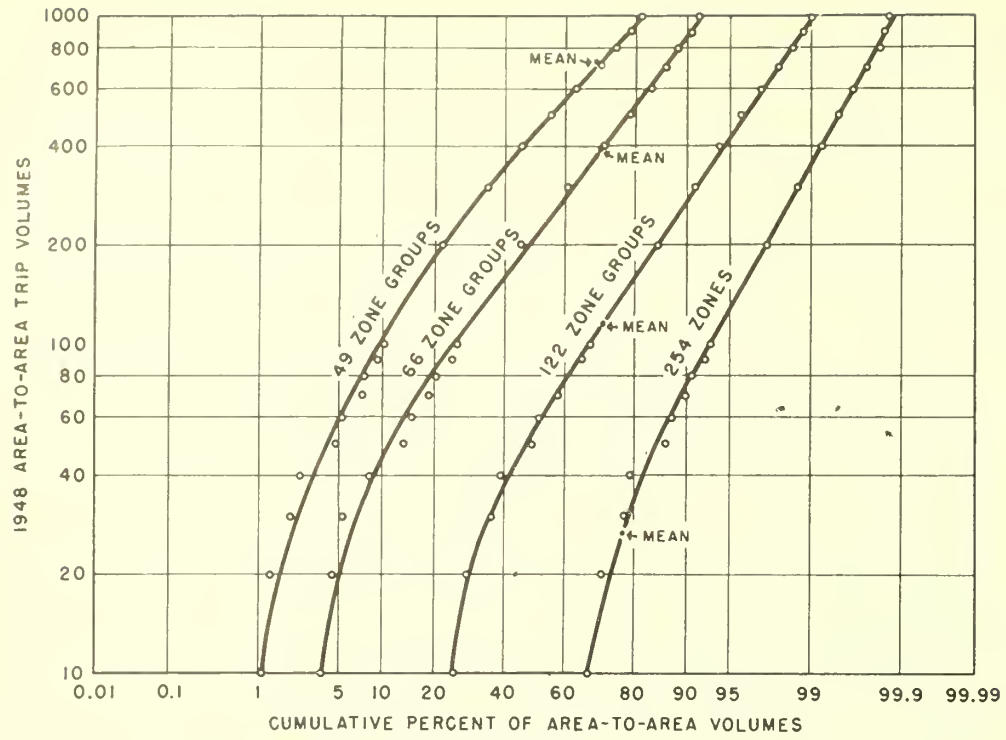

Figure 5.—Frequencies of 1948 area-to-area trip volumes of passenger cars for various zone groupings.

the square root of the sum of the squares of the two errors to determine the total error. The results of this test are shown in figure 4.

Assuming that this chart has some validity with reference to the problem, it indicates that the error for a volume of 10,000 trips is within acceptable limits and that it does not make too much difference what size zone group is used, although a minimum error was obtained for the zone grouping which had 10,000 trip-ends per group. This conclusion is dependent, to a substantial degree, on statistical inference and should be subjected to an actual test before it can be fully accepted.

Distribution of Zone-to-Zone Volumes

Even though the accumulated volumes of 10,000 or more apparently will have errors of rather modest proportions, it is desirable to inquire into the reasons for the inaccuracies of the movements between zones as they were originally planned and subsequently enlarged.

The test program was set up to count the number of zone-to-zone movements in each of several 1948 volume classes as previously described. This procedure was followed for the original 254 zones and for the subsequent groupings of 122, 66, and 49 zones.

The results of this test are shown in figure 5 for passenger cars (including taxis). Note that about 93 percent of the movements between the 254 zones have a volume of less than 100. When the number of zones was reduced to 122, about two-thirds of the area-to-area movements were less than 100; with 66 zones about one-fourth of the movements were less than 100; and with 49 zones about 10 percent were less than 100. Also note that the number of area-to-area movements which are less than the mean exceeds the number that are greater than the mean, which shows that the distribution is skewed. This is true for each of the zone groups although slightly less pronounced as the number of zones is successively decreased.

The test program also permitted the determination of the percentage of 1955 trips that were accumulated in each of the 1948 volume classes. The results of this test are shown in figure 6. Note that 50 percent of the 1955 passenger-car trips were made between zone pairs that in 1948 had a volume of less than 100 passenger-vehicle trips per day. Values for other zone groups and other 1948 volumes can be read from the chart.

In summation then, the preponderance of zone-to-zone movements within a metropolitan area is exceedingly small but because of the large number of such movements, they do account in the aggregate for a substantial portion of present or predicted trips.

Prediction Error Related to Zone-to-Zone Volumes

There is, of course, no *apriori* reason why small zone-to-zone volumes cannot be forecast with accuracy equal to large zone-to-zone volumes. The converse that the forecast error should be independent of the volume would likely seem true.

However, there is *apriori* probability that the error in the expanded number of trips from a sample survey is inversely proportional to the number of sample trips interviewed as subsequently shown.

Of all trips into and out of zone i there is a certain proportion p that will be from or to zone j. Therefore, p is the proportion of trip-ends in zone i with the other end of the trip in zone j, and $1-p$ (equals q) is the proportion of trip-ends in zone i that do not have the other end of the trip in zone j.

If s trips with an end in zone i are reported by interview, the probable number $\bar{x}$ with the other end in zone j is $s \times p$. Mathematically $\bar{x}=sp$.

From any standard statistical text it can also be shown that the standard deviation σ of the number of trips reported between zone i and zone j from sample interviews made

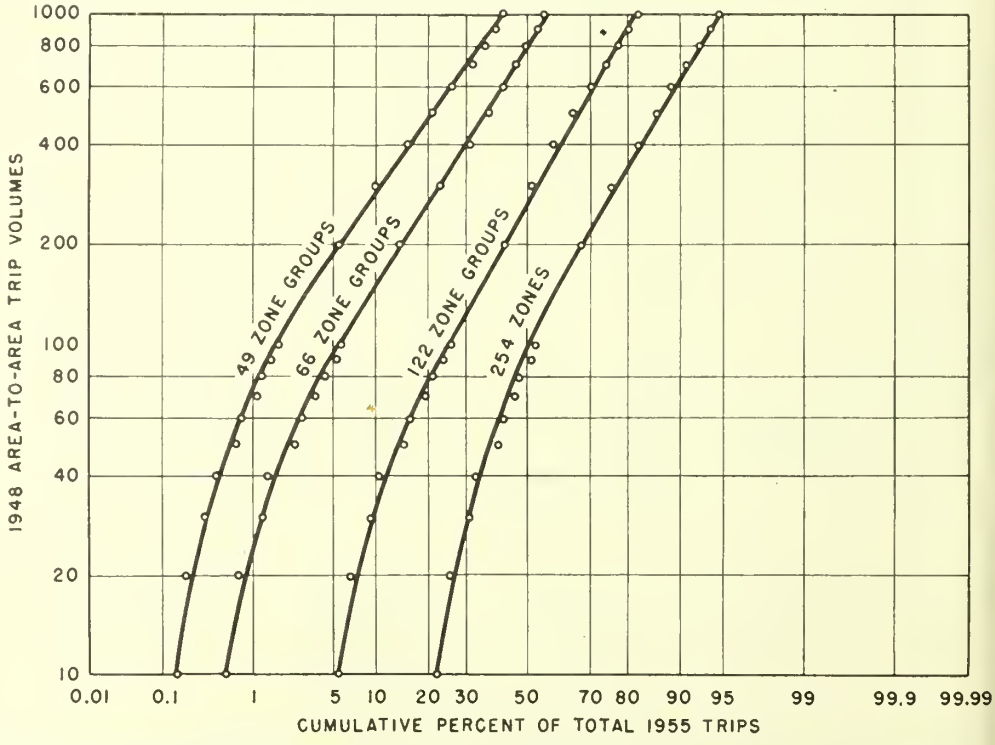

Figure 6.—Percentage of 1955 passenger-car trips that were accumulated in each of the 1948 area-to-area trip volume classes.

Table 7.—Percentage root-mean-square error by volume class of zone-to-zone movements of passenger vehicles

1948 volume class	254 zones	122-zone groups	66-zone groups	49-zone groups
20–29 [1]	195.0	121.0	95.6	106.1
30–39	222.1	101.4	86.7	83.4
40–49	279.7	118.1	81.5	73.4
50–59	141.0	96.3	82.3	89.4
60–69	139.9	107.5	92.6	89.7
70–79	108.4	74.2	78.2	76.2
80–89	154.3	95.2	94.9	54.4
90–99	128.2	83.0	54.8	73.2
100–199	133.4	93.3	64.8	64.8
200–299	80.5	69.7	58.6	52.1
300–399	60.7	64.0	56.0	43.8
400–499	54.9	56.6	50.9	38.4
500–599	42.6	42.3	45.9	44.1
600–699	51.2	40.6	46.7	48.6
700–799	88.8	41.1	31.1	26.5
800–899	38.9	26.5	57.2	31.6
900–999	18.4	21.4	33.1	38.4
1,000 and over	28.6	27.5	21.2	25.3

[1] Because of home interview sampling ratio of 1:20 in 1948, errors for volume classes below 20 cannot be accurately appraised.

of trips with one end in zone i is $\sqrt{spq}$. Mathematically $\sigma=\sqrt{spq}$.

The standard deviation σ as a proportion of the expected number $\bar{x}$ is $\sigma/\bar{x}=\sqrt{spq}/sp=\sqrt{q/sp}$.

Since p and q are constants for any pair of zones, the percentage error varies inversely with the square root of the number of trips between zone i and zone j obtained by interview. For 254 zones p will have an average value of 1/254 and q, 253/254. Thus,

$$\frac{\sigma}{\bar{x}}=\sqrt{\frac{\frac{253}{254}}{\frac{s}{254}}}=\sqrt{\frac{253}{s}}$$

Again the average zone had 6,900 trip-ends in 1948, of which about 345 were obtained by interview, so on the average $s=345$. Thus, $\sigma/\bar{x}=\sqrt{253/345}=0.86$. For an average movement in 1948, a standard deviation error of 86 percent could be expected.

Since the test of forecasting procedures uses the reported 1948 trips as the base data for calculating the 1955 trips and then compares the result with the reported 1955 trips, it would be expected that the error would be increasingly large for the smaller trip volumes, not because of errors in the forecasting procedure but due to sample variability.

The RMS error in the various 1948 volume classes for passenger vehicles in the seventh approximation by the Fratar method behaves in this manner as is shown in table 7 for each of the zone groups.

Recommended Procedures

The primary purpose of forecasting zone-to-zone volumes is for the selection and assignment of trips to a transportation network. To do this with reasonable accuracy, particularly at each ramp on an expressway network, it is imperative that the zones be of a size consistent with the distance between ramps. Thus, while increasing the size of the zone increases the accuracy of predictions of the zone-to-zone movements, this procedure adversely affects the primary purpose of forecasting. Pending further studies as outlined at the end of this article, it is recommended that zones be established in accordance with present practice and that the zone-to-zone movements be forecast by the Fratar method.

A flow chart for accomplishing this forcast on an electronic computer is shown in figure 7. This diagram is made up for an input of zone-to-zone volumes as file A and a combination input of present trip-ends, future trip-ends, and growth factors by zones as alternate inputs for file B. The program includes appropriate editing routines for checking the present trip-ends and the growth factors, if these are available independently from the zone-to-zone volumes in file A. The switches shown on the flow chart are programed transfer points resulting from decisions made at a remote point in the program. They are not switches on the console of the computer.

A program written from this flow chart includes the computation of the frequency of the new F factors, to be used as a guide for determining the need for continuing additional iterations.

Future Research

The work to date indicates that the three iterative methods are equally accurate in computing future trips, but the Fratar method arrives at the minimum error in fewer approximations. It is also more efficient in its rate of closure.

In addition it has been found that the preponderant small volume movements are individually affected by an inherent sample variability. The summation of these movements accounts for a substantial portion of the total trips. It is also known that the small-volume zone-to-zone movements are, on the average, the longer trips within the city that account for proportionately more vehicle-miles of travel and are also the trips most likely assignable to high-type highway facilities.

Accumulating trips across a grid

However, it is not necessary that the individual zone-to-zone movements be accurate if a summation of these volumes crossing the city is reasonably representative of the actual travel. To determine this relation it is proposed that each zone-to-zone movement be traced from the X and Y coordinates of one zone to the X and Y coordinates of the other. Each time this trace intersects a predetermined section of a grid line, the number of 1955 trips for this zone-to-zone movement as projected from the 1948 data and the number as determined from the 1955 survey are "remembered." When all zone-to-zone movements have been similarly traced, the number of remembered trips over an appropriate interval of each grid line is totaled. The comparison of the total number of trips projected from the 1948 data with the total number determined from the 1955 survey will give a measure of the accuracy of the projection.

The use of the trace principle automatically weights the longer trips properly in that longer trips will cross many grid lines, whereas the short trips may cross none or only a few. Further, by choosing an appropriate length along each grid line, the summation of trips to various volumes can be accomplished.

While the trace method appears to be a difficult manual task, it is comparatively simple to program for an electronic computer and would require about the same computer time as a single iteration of the Fratar method if the number of grid lines is held to a reasonable minimum.

Stabilizing the small-volume movements

Another research project which should be undertaken is the testing of methods for stabilizing the small-volume zone-to-zone movements. One of the more difficult problems in forecasting traffic is that of predicting the future number of trips for the zone-to-zone movements that are zero at the present time, since all methods tested require the multiplication of existing trips by various factors. The magnitude of this problem can be most easily visualized by remembering that about 67 percent of all possible 1948 zone-to-zone movements in the Washington survey were zero and these same zone-to-zone movements accounted for 22 percent of all 1955 trips.

In addition to the zero volumes, other low volumes are also inherently inaccurate. For an understanding of this problem it is necessary to again resort to statistical formulas. The proportional error that may be expected in random sampling of all trips from one zone to another is given by the previously defined expression $\sigma/\bar{x}=\sqrt{q/sp}$.

For example, suppose that zone i has a total of 6,000 trip-ends. Of these 6,000 assume that 666 have their other ends distributed as follows: 600 in zone j, 60 in zone k, and 6 in zone l. With a 1 in 20 sample the 6,000 trip-ends in zone i will represent 300 trip-ends obtained by interview; therefore s equals 300. Considering all of the trip-ends in zone i, the probability of any trip-end having the other

Table 8.—Theoretical error in random sampling of small trip volumes

Zone-to-zone movements	Number of trips between zones	Number of trip-ends determined by interview s	Probability of trips from zone i ending in—		Proportional error	Percentage error
			Specified zone p	Other than specified zones q		
i to j	600	300	0.1	0.9	0.17	17
i to k	60	300	.01	.99	.57	57
i to l	6	300	.001	.999	1.83	183

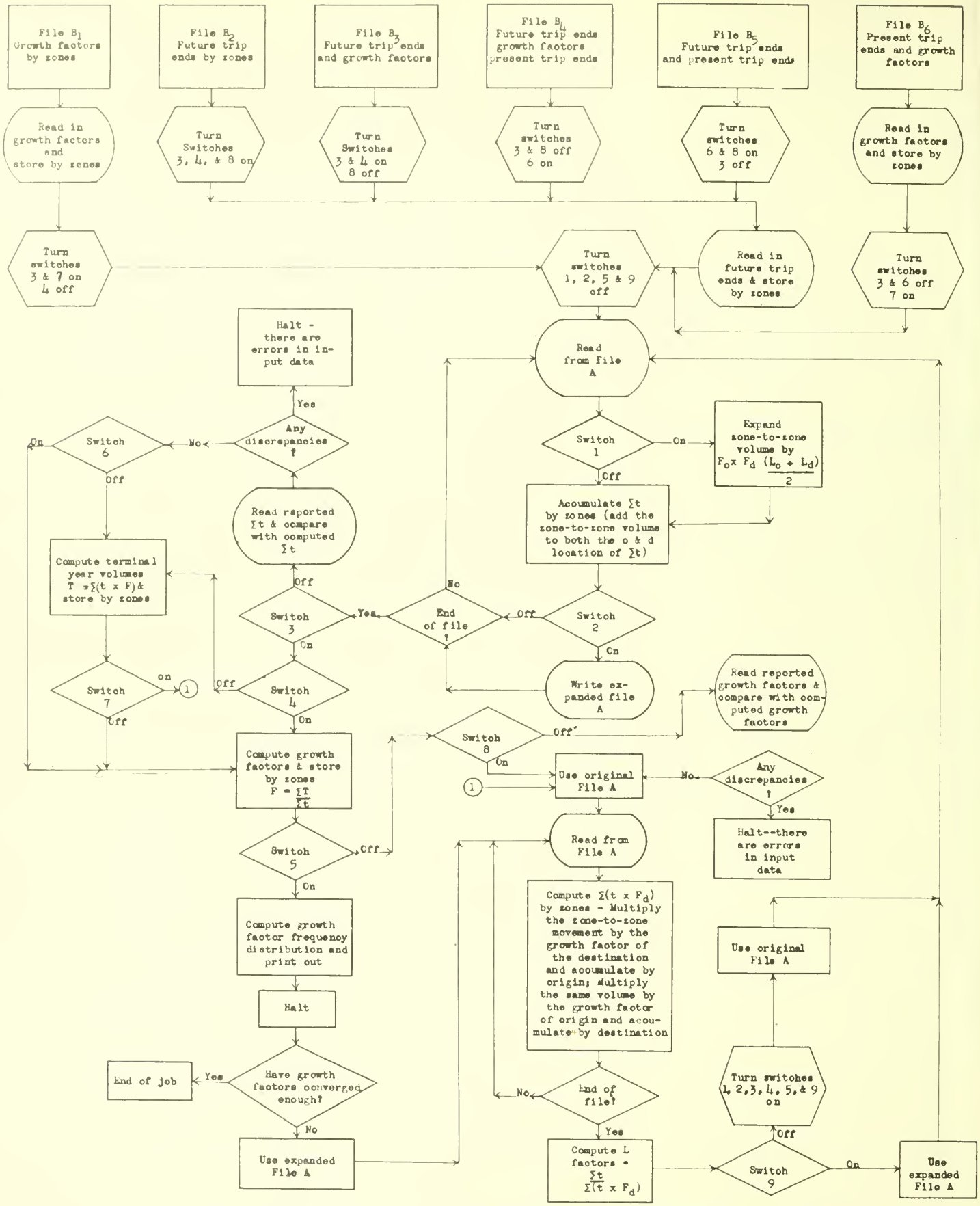
File B1 Growth factors by zones
File B2 Future trip ends by zones
File B3 Future trip ends and growth factors
File B4 Future trip ends growth factors present trip ends
File B5 Future trip ends and present trip ends
File B6 Present trip ends and growth factors
Read in growth factors and store by zones
Turn Switches 3, 4, & 8 on
Turn Switches 3 & 4 on 8 off
Turn switches 3 & 8 off 6 on
Turn switches 6 & 8 on 3 off
Read in growth factors and store by zones
Turn switches 3 & 7 on 4 off
Turn switches 1, 2, 5 & 9 off
Read in future trip ends & store by zones
Turn switches 3 & 6 off 7 on
Halt - there are errors in input data
Read from file A
Yes
Any discrepancies ?
No
On
Switch 6
Off
Switch 1
On
Expand zone-to-zone volume by $F_o \times F_d \frac{(L_o + L_d)}{2}$
Off
Read reported ∑t & compare with computed ∑t
Acoumulate ∑t by zones (add the zone-to-zone volume to both the o & d location of ∑t)
Compute terminal year volumes T = ∑(t x F) & store by zones
Off
Switch 3
Yes
End of file ?
No
Off
Switch 2
On
On
Switch 7
on
1
Off
Switch 4
On
Write expanded file A
Read reported growth factors & compare with computed growth factors
Off
Switch 8
On
Compute growth factors & store by zones $F = \frac{\sum T}{\sum t}$
1
Use original File A
No
Any discrepancies ?
Yes
Halt--there are errors in input data
Switch 5
Off
On
Read from File A
Compute ∑(t x F_d) by zones - Multiply the zone-to-zone movement by the growth factor of the destination and accumulate by origin; multiply the same volume by the growth factor of origin and accumulate by destination
Use original File A
Compute growth factor frequency distribution and print out
Halt
End of job
Yes
Have growth factors converged enough?
No
End of file?
No
Yes
Turn switches 1, 2, 3, 4, 5, & 9 on
Off
Use expanded File A
Compute L factors = $\frac{\sum t}{\sum (t \times F_d)}$
Switch 9
On
Use expanded File A

Figure 7.—Electronic computer flow chart for forecasting zone-to-zone trips by the Fratar method.

Table 9.—Theoretical distribution and probable accuracy of reported trips between zones

Reported number of trips	Probability of reported number of trips from—	
	Zone i to zone k	Zone i to zone l
0	0.05	0.74
20	.15	.22
40	.22	.03
60	.23	.01
80	.17	
100	.10	
120	.05	
140	.02	
160	.01	

end in zone j is 600 out of 6,000 or 0.1. Therefore $p=0.1$ and $q=1-p=0.9$. Thus for the trips from zone i to zone j, the proportional error $=\sqrt{q/sp}=\sqrt{0.9/300\times 0.1}=0.17$. Thus with a one-twentieth sample, a standard deviation accuracy of 17 percent for the trips between zone i and zone j could be expected. Similar values for zones k and l are shown in table 8.

If the sample rate is 1 in 20, it is manifestly impossible that the 6 trips between zone i and zone l will ever be reported as 6 trips. However, from the expansion of the binomial $(p+q)^s$, the frequency distribution of the reported trips between zone i and zone k and between zone i and zone l is estimated as shown in table 9.

Thus the 6 trips between zone i and zone l are never correctly reported; 74 percent of the time they are reported as zero; and the remaining 26 percent of the time they are reported as 20 or more trips. Similarly the 60 trips between zone i and zone k are reported as 60 only 23 percent of the time, while the remaining 77 percent of the time the reported trips are in error by more than 33 percent.

Adjacent to zone i there will be other zones i_2 and i_3. To illustrate, it can be assumed that zone i_2 has 8,000 trip-ends and zone i_3 has 6,000 trip-ends; therefore since zone i has 6,000 trip-ends, in zones i, i_2, and i_3 there would be a total of 20,000 trip-ends which would represent 1,000 interviews.

Now since zones i_2 and i_3 are adjacent to zone i, it is probably true that their movements to zone l are similar in volume to the movements from zone i to l. This assumption is justified by the high correlation of distance and trip volume as established by Dr. Carroll and others. If this be true, it can be assumed that 1/1,000 of all trip-ends in zone i_2 and i_3 are to or from zone l as was the case with zone i; the 20 trips between zone l and the three i zones are divided 30 percent to zone i, 40 percent to zone i_2, and 30 percent to zone i_3. If the expansion process $(p+q)^s$ is again used to obtain the probability of the zone-to-zone movements, the results are as shown in table 10.

From a comparison of tables 9 and 10, it appears that grouping of zones will improve the accuracy of the low-volume movements. This process can be continued for other groupings. However, instead of using this rather time-consuming computation we can obtain the proportional error by the simpler equation, $\sigma/\bar{x}=\sqrt{q/sp}$, which does not indicate the frequency of the various movements but does, in one operation, compute the resulting error.

The trips between zones i, i_2, and i_3 combined and zone l will have a proportional error of $\sigma/\bar{x}=\sqrt{0.999/1{,}000\times 0.001}=\sqrt{0.999}=1.00$ or 100 percent. Thus by grouping three zone-to-zone movements the standard-deviation error has been reduced from 186 to 100 percent, assuming that the distribution of trips between zone l and zones i, i_2, and i_3 is proportional to the distribution of trip-ends in the three i zones. It is true, of course, that this assumption will not be exactly correct, but it seems likely that the error of this assumption will be less than the error added by the individual zone sample variability.

The problem of how large a grouping is desirable can be approximated from the equation $\sigma/\bar{x}=\sqrt{q/sp}$. Note that the limiting value of q is 1.00. It can never be as large as 1.00, and in almost all applications it is not smaller than 0.9. At the same time sp is equal to the number of trips, obtained by interview, between a pair of zones with perfect sampling. The value of sp can range from zero up to the value of s. The relation between the error of a zone-to-zone movement and the number of trips, determined by interview, making this zone-to-zone movement can be approximated as follows:

Number of trips	*Percentage error*
0	∞
1	100
2	71
5	45
10	32
15	26
20	22
30	18

Considering the undesirability of combining too many zones, from this tabulation it appears that zone-to-zone movements might well be grouped until the accumulated movement represents about 10 interviews.

A method to accomplish this purpose has been worked out but not tested, except by hand computation from a small sample. The sample computation indicated that the error is reduced by approximately one-third. The method requires the use of the binary system in coding a group of zones. For illustrative purposes a group of 4 zones can be considered, although in actual practice probably 16 would be required.

To illustrate, suppose that in region A the area is divided with 2 zones in each half. One-half of the zones would be designated *A0* and the other half *A1*. These pairs of zones would again be divided in half or into single zones designated *A00*, *A01*, *A10*, and *A11*. A separate region B would be similarly separated into *B00*, *B01*, *B10*, and *B11*.

Table 10.—Theoretical improvement in the accuracy of low-volume trip movements by zone grouping

Reported number of trips to zone l from—				Probability
Zone i	Zone i_2	Zone i_3	Total	
0	0	0	0	0.37
6	8	6	20	.37
12	16	12	40	.18
18	24	18	60	.06
24	32	24	80	.02

Now if only trip volumes that represent 10 interviews (or 200 trips with a 1/20 sample) are considered sufficiently stable so as not to warrant readjustment, a method of combining zones that is amenable to computer operation is required. A suitable method is as follows:

The number of trips from *A00* to *B00* is examined. If it is less than 200 (10 interviewed trips), combine zones *B00* and *B01* and find the number of trips between *A00* and *B00+B01*. If it is still less than 200, combine zones *A00* and *A01* and find the number of trips between *A00+A01* and *B00+B01*. If the number of trips is still less than 200, again double the B area to include 4 zones, and if necessary, double the A area to include 4 zones, and so on until the figure 200 is reached.

The advantage of the binary coding is that it provides the desired grouping through a simple arithmetic operation. The arithmetic operation simply combines the binary portion of the region code by alternate digits. For example, if *A00* is written as AA_1A_2, and *B00* is written as BB_1B_2 the combination is written $A_1B_1A_2B_2$. Starting with 0000, the digit 1 is added successively until the sum 1111 is obtained. The subtotals are then decoded by the $A_1B_1A_2B_2$ pattern and the zone-to-zone movements are then in the proper order for combining.

From the original zone-to-zone volumes and the combination totals, the preselected volume can be determined. The combination volume is then reassigned to individual zone-to-zone movements. Suppose that in the previous example all 4 of the A zones were combined and all 4 of the B zones were combined to provide a preselected volume. The total trip-ends in the individual zones in the A group are added and the proportion of the total in each A zone is determined (P_{A1}, P_{A2}, etc.). Similarly each zone of the B group is a certain proportion of the B total trip-ends (P_{B1}, P_{B2}, etc.). Then the total volume V between the groups is reassigned to individual zone-to-zone volumes, V_{A1-B1}, by the equation $V_{A1-B1}=V\times P_{A1}\times P_{B1}$, $V_{A1-B2}=V\times P_{A1}\times P_{B2}$, etc.

From sample tests made to date, this reassignment procedure appears to improve the accuracy of predicting the future zone-to-zone trip movements. Whether it improves the accuracy of predicting accumulated volumes such as would occur on road sections or ramps should be tested by the process of accumulating the trips across a grid, as previously described.

Factors Affecting Trip Generation of Residential Land-Use Areas

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported [1] by GORDON B. SHARPE, Head, Urban Highway Section, WALTER G. HANSEN, Highway Research Engineer, and LAMELLE B. HAMNER, Analytical Statistician

The present and potential trip-generating power of urban residential areas can be estimated with a reasonable degree of accuracy, dependent upon the availability and reliability of certain related information. The major factors affecting trips by residents of an area are population and automobile ownership. With data normally developed from a home-interview-type origin and destination survey it is possible either to up-date residents trip information or, with slightly less reliability, to forecast the trips for some future date.

It was found that the total number of trip-ends in a residential area is approximately equal to the number of trips made by residents of that area between all origins and destinations. Therefore, any method for estimating residents trips is equally applicable to estimates of total trip-ends on residential land. As a corollary, the number of trip-ends to nonresidential land in an urban area can be estimated if the number of trips made by residents of the entire urban area are known.

Generally, about 80 percent of all trips made by residents of a residential area begin or end at home. Also approximately 80 percent of the total trip-ends on residential land, by residents and nonresidents, are home-oriented. These proportions are greater in areas where automobile ownership and economic status are low, and where population density is high. They do not vary appreciably with distance from the central business district.

These findings are based upon analyses of data from the two home-interview-type origin and destination traffic surveys made in the Washington, D. C., metropolitan area in 1948 and 1955. All results pertain to the Washington, D. C., area during the time interval studied, but it may reasonably be assumed that certain aspects of the findings and methodology will be applicable elsewhere.

SINCE 1944, comprehensive home-interview origin and destination studies have been made in well over 100 metropolitan areas. The existing pattern of travel by different means of transportation in these areas has been developed with considerable accuracy. This information has been invaluable in determining the present need for highways and transit facilities but, in itself, does not give the answer to future needs, which must be the basis for intelligent planning.

One of the basic keys to forecasting transportation needs is the establishment of the relations between travel and the type, intensity, and interrelation of land uses. When these relations have been established and trends observed, it will be possible to determine more accurately the transportation needs that will exist when anticipated urban development has taken place.

Data from a number of surveys have been analyzed to develop factors for estimating the number of trips attracted to and generated by land-use developments of different kinds, size, and intensity. These relate to specific places and times. Information on trends is comparatively meager, repeat studies having been completed in only a very few metropolitan areas. It has not, therefore, been possible to establish relations which will permit the accurate predetermination of the number of trip-ends to be expected in different areas under conditions anticipated for some time in the future, these being dependent upon the stability of a number of factors.

In the Washington, D. C., area, a home-interview type of origin and destination survey was made in 1948. A repeat study was made in 1955. In the earlier survey a 5-percent sample was obtained by interviewing the residents of 1 of every 20 dwelling units. In the latter survey, the sample rate was 1 in 30 in the District of Columbia and 1 in 10 in the Maryland and Virginia suburban areas. These two surveys provide a basis for studying factors and trends relating to urban travel habits, and several research projects have been carried out with this objective.

Scope of Study

This article is concerned with the evaluation of methods for estimating the potential generation and production of trips in urban residential developments. Another article entitled "Evaluating Trip Forecasting Methods with an Electronic Computer", which appears on pages 77–87 of this issue, is concerned with the evaluation of methods of estimating the distribution of zone-to-zone trip movements resulting from zone growth.

There are many factors, other than size, that could be expected to affect the potential generation and production of trips in urban residential developments. This article presents the results of an analysis of the effect that differences in population, automobile ownership, income per household, distance from the central business district, and population per net residential acre had on the number of trips attracted to and generated by residential land. These factors were selected because it has been possible to establish measures that are free from personal bias, and it seems that they constitute a logical premise on which to begin an analysis of total trips generated by residential land-use areas.

Because the summarization of the 1955 data has not progressed far enough to permit a study by mode of travel, the current analysis is confined to total trips to and from residential land by all modes of travel, except trips by taxi and truck operators in the course of their daily work and pedestrian trips.

Sources of Information

The trip data obtained from the Washington transportation surveys of 1948 and 1955, upon which this study is based, reflect total trips by persons for an average weekday of the respective survey periods. Data on population, automobile ownership, and distance from the central business district were also obtained from these surveys.

The 1955 family income information was obtained from the National Capital Regional Planning Council. Residential acreage figures were developed jointly by the staffs of the Washington Metropolitan Area Transportation Study, which made the 1955 survey, and the National Capital Regional Planning Council. The data developed were net residential acres; streets and all nonresidential land uses were excluded.

This article combines the results of a number of studies based upon travel data obtained from the two Washington transportation surveys. The basic data were summarized by groupings best suited for the objectives under

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

Table 1.—Characteristics of 10 selected residential areas included in the 1948 Washington, D. C., metropolitan area survey

Area number	Name of area	Predominant residence type	Economic class [1]	Central business district to residence		Number of dwelling units	Number of passenger cars owned	Population	Persons per dwelling unit	Passenger cars per dwelling unit	Automobile driver trips per passenger car
				Travel distance	Average peak-hour driving time, 1950						
				Miles	*Minutes*						
1	Massachusetts Avenue Park, D. C.	Single family detached	Very high	3.29	12	158	180	518	3.28	1.14	2.55
2	Wesley Heights and Spring Valley, D. C.	do	Very high and upper high.	5.29	16	712	1,201	2,896	4.07	1.69	3.83
3	American University Park, D. C.	Single family detached [2]	High	5.71	18	2,375	2,039	7,975	3.36	.86	4.20
4	Beverly Hills, Braddock Heights, and Del Ray, Va.	do [2]	Above average	7.43	17	1,426	1,447	5,176	3.63	1.01	4.84
5	Fairlington and Parkfairfax, Va.	Garden-type apartments	do	6.86	15	4,826	3,833	15,658	3.24	.79	4.63
6	Brightwood area, D. C.	2-family semidetached and apartments.	do	5.29	18	2,497	2,209	9,354	3.75	.88	3.77
7	U. S. Soldiers Home area, D. C.	Multifamily apartments and row houses.	Below average	2.71	12	2,101	1,404	6,913	3.29	.67	2.67
8	Shirley Homes, Va.	Government low-rent row houses.[2]	do	5.71	13	1,352	686	4,805	3.55	.51	4.21
9	Central city area, D. C.	Row houses	do	1.14	6	3,270	582	10,977	3.36	.18	1.95
10	Southwest area, D. C.	Single family detached and row houses.	Low and very low	1.14	7	6,596	1,975	28,657	4.34	.30	1.98
Total or average, 10 areas						25,313	15,556	92,929	3.67	.61	3.75
Total or average, metropolitan area						336,181	203,464	1,109,860	3.30	.61	3.10

[1] Source: *District of Columbia residential areas*, Economic Development Department, Board of Trade, Washington, D. C.; *Virginia residential areas*, estimated on the basis of comparable rentals, housing values, and income reported in the 1950 Censuses of Population and Housing. Economic classes are based on 1949 estimated income as follows: very high, $10,200 and over; upper high, $8,500–$10,199; high, $6,800–$8,499; above average, $5,100–$6,799; below average, $3,825–$5,099; low, $2,550–$3,824; and very low, under $2,550.
[2] Includes small number of apartment units.

Table 2.—Residents average weekday trips, by purpose and mode of travel, recorded in the 1948 Washington, D. C., metropolitan area survey

Area number	Resident trips per dwelling unit by purpose										Resident trips per dwelling unit by mode of travel			
	Work	Business	Medical and dental	School	Social and recreational	Change travel mode	Eating	Shopping	Serving passengers	Home	All purposes	Automobile driver	Automobile and taxi passengers	All automobile trips	Mass-transit passengers
1	1.02	0.87	----	0.15	0.58	0.58	0.15	0.58	0.30	2.17	6.40	2.90	1.75	4.65	1.75
2	2.42	.51	0.07	.36	1.98	.20	.48	.82	.58	4.60	12.02	6.45	3.36	9.81	2.21
3	1.32	.55	.06	.25	1.48	.08	.13	.75	.48	3.23	8.33	3.60	2.89	6.49	1.84
4	1.49	.33	.03	.34	.99	.11	.13	.93	.96	3.31	8.62	4.91	2.25	7.16	1.46
5	1.36	.12	.10	.33	1.00	.09	.15	.90	.52	2.97	7.54	3.68	2.22	5.90	1.64
6	1.95	.30	.06	.24	.91	.15	.18	.80	.29	2.75	7.63	3.34	1.57	4.91	2.72
7	1.62	.14	.09	.12	.53	.01	.10	.28	.11	2.45	5.45	1.78	1.11	2.89	2.56
8	1.06	.08	.03	.09	.74	.06	.11	.63	.31	2.12	5.23	2.14	1.52	3.66	1.57
9	1.23	.16	.04	.03	.17	.01	.01	.08	.01	1.54	3.28	.35	.47	.82	2.46
10	1.21	.14	.03	.08	.22	.003	.02	.11	.02	1.59	3.43	.59	.40	.99	2.44
Average, 10 areas	1.40	.22	.06	.17	.70	.06	.10	.50	.28	2.40	5.89	2.31	1.42	3.73	2.16
Average, metropolitan area	1.32	20	.05	.12	.62	.07	.09	.33	.19	2.13	5.12	1.88	1.22	3.10	2.02

consideration. For example, one very limited study utilized data for only 10 residential areas. A subsequent study was confined to the District of Columbia for which data on family income and population density were available by census tracts. Another study was based, in part, on the analysis of 118 subzones throughout the survey area that could be classed as purely residential; and, in part, on 77 of the 118 subzones grouped into 48 residential areas, each of which had 1,000 or more trip origins. Then again, tests were made using 56 of the 118 subzones grouped into 35 residential areas, each of which had 2,000 or more trip origins. Finally, an analysis was made of trips by residents of 200 zones selected so that data from the surveys would be comparable. These zones contained 96 percent of the population residing within the 1948 traffic study cordon and 93 percent of the population living within the 1955 cordon.

Table 3.—Basic data for 118 selected residential subzones in the 1955 Washington, D. C., survey area, grouped according to distance from the central business district

Rings [1]	Number of—							
	Subzones	Interviews	Residents	Dwelling units	Automobiles	Net residential acres	Residents trips [2]	Automobiles per 100 residents
2	2	46	4,689	1,820	1,182	9.5	7,574	25.2
3	3	42	4,603	1,602	881	141.8	6,853	19.1
4	16	282	17,987	6,412	5,251	487.4	32,988	29.2
5	30	379	27,014	8,817	9,305	956.3	56,973	34.4
6	25	348	18,584	5,742	6,480	782.5	39,636	34.9
7	16	454	17,735	5,190	6,242	1,041.2	39,481	35.2
8	12	279	10,727	3,007	3,657	536.8	23,386	34.1
9	12	229	9,407	2,527	3,278	1,259.2	20,793	34.8
10	2	68	3,086	798	962	215.9	6,185	31.2
Total	118	2,127	113,832	35,915	37,238	5,430.6	233,869	32.7

[1] Distance (miles) of residence from the central business district. None of the selected subzones were in rings 1, 11, 12, or 13. [2] On an average weekday in 1955.

Conclusions

The results of the analysis indicate a means of estimating residents trips and trip-ends on residential land with a reasonable degree of accuracy. It would appear desirable to ascertain and develop additional factors by which the accuracy of the technique could be greatly improved in each specific zone. It would also be desirable to continue these studies, where possible, over a longer period of time to determine whether factors such as trips per person remain constant beyond a 7-year interval. In addition, similar studies are needed in other urban areas of different sizes and with different travel characteristics; for instance, in an area where mass-transit facilities are much less prevalent or negligible.

The use of population data as estimating

(Continued on page 92)

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHIC

Prepared by the

Line	State	Width inches[1]	Height ft.-in.	Length—feet[2]				Number of towed units[3]			Axle load—pounds				T
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	[6]12-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Ta
2	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Ta
3	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7]18,500	32,000	32,500	Sp
4	California	96	13-6	35	[9]35	[10]60	60	NR	NR	NR	18,000		32,000		Ta
5	Colorado	[11]96	[12]13-6	35	40	60	60	1	2	2	18,000		36,000		Fo
6	Connecticut	102	12-6	45	45	45	NP	1	NP	NP	22,400	22,848	36,000	36,720	Sp
7	Delaware	96	[6]12-6	35	42	50	60	1	1	2	20,000		36,000		Ta
8	District of Columbia	96	12-6	35	35	50	80	1	1	NP	22,000		38,000		Ta
9	Florida	96	[6]12-6	[14]35	40	50	50	1	1	NP	20,000	22,000	40,000	44,000	Ta
10	Georgia	96	13-6	[15]+39	[15]+45	48	48	1	1	NP	18,000	20,340	36,000	40,680	Sp
11	Hawaii	[16]108	13-0	40	40	55	65	1	1	2	24,000		32,000		Fo
12	Idaho	[16]96	14-0	35	[19]40	60	65	1	1	2	[17]18,000		[20]32,000		Ta
13	Illinois	96	13-6	42	42	50	50	1	1	2	18,000		32,000		Sp
14	Indiana	96	13-6	36	40	50	50	1	1	2	[23]18,000	[23]19,000	[23]32,000	[23]33,000	Sp
15	Iowa	96	[6]12-6	35	[19]40	50	NP	1	NP	NP	18,000	18,540	32,000	32,960	Ta
16	Kansas	96	12-6	35	[19]40	50	50	1	1	NP	18,000		32,000		Ta
17	Kentucky	96	[25]13-6	[26]35	[26]35	[27]50	NP	1	NP	NP	18,000	[28]18,900	32,000	[28]33,600	Sp
18	Louisiana	96	[6]12-6	35	[19]40	50	60	1	1	NP	18,000		32,000		Ax
19	Maine	96	[30]12-6	50	50	50	50	1	1	NP	22,000		32,000		Ta
20	Maryland	96	[6]12-6	55	55	55	55	NR	NR	NR	22,400		[31]40,000		Fo
21	Massachusetts	96	NR	35	[19]40	45	NP	1	NP	NP	22,400		36,000		Ta
22	Michigan	96	13-6	35	40	55	55	1	1	2	[33]18,000		[34]32,000		Ax
23	Minnesota	96	[6]12-6	40	40	50	50	1	1	NP	18,000		32,000		Ta
24	Mississippi	96	[6]12-6	35	40	[36]50	50	1	1	NP	18,000		28,650	[35]32,000	Ta
25	Missouri	96	12-6	35	40	50	50	1	1	2	18,000		32,000		Ta
26	Montana	[18]96	13-6	35	40	60	60	1	1	[37]2	18,000		32,000		Ta
27	Nebraska	96	13-6	35	[19]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Ta
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Ta
29	New Hampshire	96	13-6	35	[35]40	45	45	NR	NR	NR	22,400		36,000		Ta
30	New Jersey	96	13-6	35	[39]35	45	[40]50	1	1	NP	22,400	23,520	32,000	33,600	Sp
31	New Mexico	[41]96	13-6	40	40	65	65	1	1	2	21,600		34,320		Ta
32	New York	96	13-0	35	[42]35	50	50	1	1	NP	22,400		36,000		Fo
33	North Carolina	96	[6]12-6	35	[19]40	[43]50	[43]50	1	1	NP	18,000	19,000	36,000	38,000	Sp
34	North Dakota	96	13-6	[14]35	[19]40	50	50	1	1	NP	18,000		30,000		Fo
35	Ohio	96	3-6	35	[19]40	50	60	1	NR	NR	19,000	20,000	[44]31,500	32,500	Fo
36	Oklahoma	96	13-6	35	45	50	50	1	1	NP	18,000		32,000		Ta
37	Oregon	96	[45]12-6	35	[35]40	[46,35]55	[35]60	1	1	[35]2	[47]18,000		[47]32,000		Ta
38	Pennsylvania	96	[5]12-6	35	[19]40	50	[40]50	1	1	NP	22,400	23,072	36,000	37,080	Sp
39	Puerto Rico	96	12-6	NS	NS	NS		NS	NS	NS	NS		NS		Sp
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		NS		Sp
41	South Carolina	96	[54]12-6	[14]35	[19]40	50	60	1	1	NP	20,000		32,000		Ta
42	South Dakota	96	13-0	35	40	50	60	1	1	NP	18,000		32,000		Ta
43	Tennessee	96	12-6	35	40	45	45	1	[24]1	NP	18,000		32,000		Ta
44	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Ta
45	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		33,000		Ta
46	Vermont	96	12-6	50	50	50	50	1	1	NP	NS		NS		Sp
47	Virginia	96	[5]12-6	35	[35]40	50	50	1	1	NP	18,000		[56]32,000		Ta
48	Washington	96	[5]12-6	35	[19]40	60	60	1	1	[57]2	18,000	[58]18,500	32,000	[58]33,000	Ta
49	West Virginia	96	[5]12-6	35	[19]40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Ta
50	Wisconsin	96	[6]12-6	35	40	50	50	1	1	NP	18,000	[60]19,500	30,000	32,000	Ta
51	Wyoming	96	13-6	40	40	60	60	1	1	2	18,000		32,000	[62]36,000	Ta
	AASHO Policy	96	12-6	35	[19]40	50	60	1	1	NP	18,000		32,000		Ta
Number of States	Higher	3	26	16	29	15	7	6	8	21	31		29		
	Same	48	25	35	18	30	12	45	38	30	20		16		
	Lower	0	0	0	4	6	32	0	5	0	0		5		

NP—Not permitted. NR—Not restricted. NS—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Limited by 40-foot maximum length of semitrailer to 55-foot practical maximum length in combination.

[11] Buses 102 inches.

[12] On designated highways; 12 feet 6 inches on other highways.

[13] Legal limit 60,000 pounds.

[14] Three-axle vehicles 40 feet.

[15] Truck 39.55 feet; bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads

[17] 700 (L+40) when L is 18' or less; 800 (L structures with span of 20' or over.

[18] Buses 102 inches on highways of surface

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timb cultural products including livestock: singl vehicle with 3 or 4 axles permitted 66,000 p permitted 79,000 pounds maximum at 43-foot a

[21] On designated highways: 16,000 pounds on

[22] Without tandem axles 45,000 pounds.

[23] On designated highways: single axle 22,4 total of all excesses of weight under one or

[24] Limited to 3,500 pounds.

[25] Class AA highways: 12 feet 6 inches on

[26] On designated highways: trucks 26.5 feet

[27] Class AA highways: 45 feet on other high

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways

[30] Including load 14 feet.

[31] Spaced less than 48 inches 36,000 pounds

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet fro

[34] On designated highways only and limited

[35] On designated highways only.

[36] Auto transports permitted 50 feet.

[37] Semitrailer and semitrailer converted to

[38] Dual-drive axles: otherwise 40,000 pound

WEIGHTS COMPARED WITH AASHO STANDARDS

oads, July 1, 1958

weight limit			Specified maximum gross weight—pounds[a]						Practical maximum gross weight—pounds[b]						Line
	Applicable to:		Truck		Truck-tractor semitrailer				Truck		Truck-tractor semitrailer				
Formula or equation	Any group of axles	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	
		X							27,800	47,600	47,600	60,010	64,650	NP	1
.	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	2
	Under 18'	Over 18'							26,500	40,500	45,000	59,000	65,000	65,000	3
									26,000	40,000	44,000	58,000	72,000	76,000	4
800 (L+40)		X	30,000	46,000					26,000	44,000	44,000	62,000	76,000	76,000	5
			32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	6
	X		30,000	46,000	48,000	60,000	60,000	60,000	28,000	48,000	48,000	56,350	60,000	60,000	7
	X								30,000	46,000	52,000	58,450	61,490	64,650	8
.		X							30,000	52,000	52,000	65,200	71,115	71,115	9
								63,280	28,340	48,680	48,680	63,280	63,280	63,280	10
& 800 (L+40)	Under 18'	Over 18'							32,000	38,800	56,000	62,800	69,600	78,000	11
	X								26,000	40,000	44,000	58,000	72,000	76,800	12
			36,000	[22]41,000	45,000	59,000	68,000	72,000	26,000	40,000	44,000	58,000	68,000	72,000	13
								72,000	27,000	41,000	45,000	59,000	73,000	73,000	14
	X								26,540	40,960	45,080	59,500	70,716	NP	15
	X								26,000	40,000	44,000	55,470	63,890	63,890	16
			36,000	50,000	54,000	59,640	59,640	NP	26,900	41,630	45,800	59,640	59,640	NP	17
									26,000	40,000	44,000	58,000	72,000	76,000	18
	X		32,000	50,000	50,000	60,000	60,000	60,000	30,000	40,000	52,000	60,000	60,000	60,000	19
850 (L+40)		X					65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	20
1,000 (L+25)		X	[32]46,000	[32]60,000	[32]60,000	[32]60,000	[32]60,000	NP	30,400	44,000	52,800	60,000	60,000	NP	21
									26,000	[35]40,000	44,000	[35]58,000	[37]66,000	[35]102,000	22
	X								26,000	40,000	44,000	58,000	68,000	72,500	23
	X								26,000	36,650	44,000	54,650	[35]58,000	[35]58,000	24
	X								26,000	40,000	44,000	55,470	64,650	64,650	25
	Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	26
	Under 22'	Over 22'							26,780	41,200	45,320	57,134	66,590	66,590	27
	Under 18'	Over 18'							26,900	41,600	45,800	60,000	74,000	76,800	28
		X	33,400	[38]47,500	52,800	66,400			30,400	44,800	52,800	66,400	66,400	66,400	29
			30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	30
	Under 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
34,000+850L		X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	32
& 750 (L+40)	Under 18'	Over 18'	31,500	46,200	46,200	58,800	58,800	58,800	27,000	46,000	46,000	58,800	58,800	58,800	33
800 (L+47.5)		X							26,000	38,000	44,000	56,000	63,750	63,750	34
	X								28,000	40,500	48,000	60,500	73,000	78,000	35
									26,000	40,000	44,000	58,000	66,000	66,000	36
	Under 18'	Over 18'					74,000	76,000	26,000	40,000	44,000	58,000	72,000	76,000	37
			33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	38
															39
			[50]36,000	[51]44,000	[52]50,000	[53]60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	40
	X								28,000	40,000	48,000	60,000	66,839	71,115	41
	X								26,000	40,000	44,000	58,000	72,000	73,280	42
		X							26,000	40,000	44,000	55,980	55,980	43,500	43
	X								26,900	41,600	45,800	57,844	61,340	61,340	44
	X								26,000	41,000	44,000	59,000	74,000	79,900	45
			30,000	40,000	50,000	[54]60,000	[55]60,000	[55]60,000	30,000	40,000	50,000	60,000	60,000	60,000	46
	X						[55]56,800	[55]56,800	26,000	40,000	44,000	56,800	56,800	56,800	47
	Under 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	48
	X								26,900	41,600	45,800	57,844	63,840	63,840	49
1,000 (L+26)	X								27,500	40,000	47,000	59,500	68,000	68,000	50
	X								26,000	44,000	44,000	62,000	73,000	73,000	51
(L+24)-3L[?]	X								26,000	40,000	44,000	55,470	61,490	71,900	
	18	19							29	27	29	46	36	20	
									21	19	21	3	2	0	
									0	4	0	1	12	30	

ri 56,000 pounds maximum.
n 18': 900 (L+40) on highways having no
otherwise as administratively authorized.
es, concentrates, aggregates, and agri-
tan m axle 37,800 pounds, gross weight table:
ot le spacing, vehicle with 5 or more axles
3000 pounds: tolerance of 1,000 pounds on
xle oad and gross weight.
oth r highways.
ss highways 30,000 pounds.
to 4,000 pounds.
co nation: otherwise 26,000 pounds.
o dolly.

[39] Or as prescribed by P.U.C.
[40] Exception for poles, pilings, structural units, etc., permitted 70 feet.
[41] On designated highways 102 inches.
[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[43] Including front and rear bumpers.
[44] Spaced less than 4 feet 24,000 pounds.
[45] Certain types of vehicles and commodities under special permit on designated highways up to 13 feet 6 inches.
[46] 60 feet allowed truck tractor semitrailer on 4 major interstate routes.
[47] Logging vehicles permitted 3-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.
[48] Governs gross weight permitted on highways designated by resolution of State highway commission.
[49] Single unit truck with 4 axles permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.
[53] Axles spaced 27 feet or more.
[54] Auto transports and vehicles transporting materials for Department of Defense on highways with adequate clearance 13' -6".
[55] Tandem axles on trailer equipped with adequate brakes.
[56] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[57] Under State highway commission rules.
[58] Within discretion of enforcement officer.
[59] Vehicles hauling logs permitted wheelbase and gross weight tolerances. Discretionary enforcement tolerances not included in computation of practical maximum gross weights.
[60] Axle load 21,000 pounds on 2-axle trucks hauling unmanufactured forest products.
[61] On Class A highways.
[62] Based on ruling of Attorney General.

Table 4.—Effect of distance from the central business district on the "from" purpose distribution of trips originating on residential land in the Washington, D. C., survey area on an average weekday in 1955 [1]

Rings [2]	Percentage distribution of trips [3] from—							All "from" trips	
	Work	Personal business	Serving passengers	Changing mode of travel	Social and recreational	Medical, dental, and eating	Home	Number	Percent
2	2.2	1.0	2.6	0.7	5.4	----	88.1	3,672	100.0
3	4.5	4.2	2.1	-	1.5	----	87.7	3,610	100.0
4	8.2	2.0	4.4	.5	4.8	0.6	79.5	16,598	100.0
5	4.5	2.0	5.0	.3	7.5	.7	80.0	28,076	100.0
6	5.8	1.3	4.7	1.5	4.6	1.3	80.8	19,684	100.0
7	4.8	2.7	4.7	.7	5.4	.8	80.9	18,854	100.0
8	3.8	2.0	5.3	1.0	4.8	.3	82.8	11,263	100.0
9	5.6	2.4	4.4	.4	5.3	.3	81.6	9,812	100.0
10	2.8	3.9	6.1	----	.5	.9	85.8	2,550	100.0
Average or total	5.2	2.1	4.7	.7	5.4	.7	81.2	114,119	100.0

[1] Based on study of 118 selected residential subzones.
[2] Distance (miles) of residence from the central business district. None of the selected subzones were in rings 1, 11, 12, or 13.
[3] Trips from shopping and school were not included, because they were probably the result of small amounts of nonresidential land use.

(Continued from page 89)

factors will provide reasonable estimates of total trip-ends on residential land by all modes of travel combined. However, in order to implement the design of transport facilities, separate estimates must be made of automobile-driver trips, mass-transit trips, and trips by all other modes of travel. It would seem necessary, therefore, to determine the extent of automobile ownership as a means of making these separate estimates for particular modes of travel.

Recognizing that a variable such as automobile ownership may only be a transient predictor of trips, it still presents, when known, a rather impressive possibility for improving traffic estimates. In this regard, it is strongly urged that automobile ownership or registration records be established within urban and metropolitan areas on a basis of statistical areas, such as census tracts, police precincts, postal zones, school districts, or origin and destination zonal areas. Such a system would also enable the use of license tags in traffic analyses, and in many other studies such as those connected with civil defense and market research.

Home Trips As a Measure of Total Trip-Ends

Any study of residential land-use trip generation must take into account the number of trips origins and trip destinations (trip-ends) in a particular residential area, and the number of trips made by the residents of that area (trip production).

The present study of residential land-use trip generation and residents trip production was started several years ago with an analysis of residents trips and other factors in 10 areas of varying types and income levels, as shown in tables 1 and 2. It was initially apparent from the data that residents trip production generally increased with income, except for the highest income areas. It was equally apparent that the number of residents trips varied directly with the number of passenger cars owned per dwelling unit.

To establish the degree of reliability of these findings and to evaluate the contributing influence of these and other variables on residents trip production, a series of statistical analyses was undertaken, based on 95 census tracts in Washington, D. C. The results of these studies were valuable in that they established statistically the possibility of estimating, with a relatively high degree of reliability, residents daily trip production.[2]

One of the most important byproducts of the analysis of the 10 residential areas was the apparent value of the study as a means of estimating traffic growth for urban redevelopment projects. For example, Area 10, the southwest area of Washington, is such an urban redevelopment area. Redevelopment plans call for the construction of apartments and row houses in the residential portion of the area. If it is assumed that data developed for Area 7, an apartment and row-house area, would be representative of the redevelopment area when completed, then automobile trips per dwelling unit will increase three times while mass-transit travel will remain relatively constant. This general method of estimating trips for a given land use should be of value, particularly in planning and designing highway facilities for redevelopment projects.

[2] *A study of factors related to urban travel,* by William L. Mertz and Lamelle B. Hamner. PUBLIC ROADS, vol. 29, No. 7, April 1957.

The 10-area study provided an insight into residents total trip production, but as most of the areas studied included other than residential land uses, it was not possible to definitely determine the actual trip generation of the residential land.

To provide accurate data for such an analysis, 118 purely residential subzones [3] of the 1955 Washington transportation survey were selected. These subzones represented 7.6 percent of the total number of subzones in the survey area, and 7.2 percent of the total area population, or 113,832 residents. The basic data, including trip information, for these residential subzones are summarized in table 3 by annular areas (rings) at varying distances from the central business district.

In the home-interview surveys, trip purpose was designated as purpose "from" and purpose "to" (from home to work, from work to shopping, etc.). For trips having one end in a given area, the number from a specific purpose generally equals the number to that purpose. For example, for each trip to work at a specific location, there is a corresponding trip from work. Furthermore, there would be an equal number of trips to home and from home in the 118 residential subzones studied.

Effect of distance from the central business district

The percentage distribution by trip purpose, as affected by distance from the central business district, is shown in table 4. Trips from home constituted 81.2 percent of all trips with origins in the studied subzones, and consequently home-oriented trips constituted the same percentage of all trips beginning and ending in these residential subzones.[4] This is an average figure and does not reflect any variations which might be due to economic, geographic, and demographic factors.

The data in table 4 also show that the percentage of trips which originated from home within these residential subzones did not vary appreciably as distance from the central

[3] Unless otherwise specified, all subsequent references to residential areas refer to areas entirely residential.

[4] From a study of trips to 210 residential blocks in the Detroit, Mich., area, it was found that 73.3 percent of the incoming trips were to home. From a study of trips to 35 residential zones in the 1948 Washington, D. C , survey area, it was found that 80 percent of all incoming trips were to home.

Table 5.—Effect of family income on the purpose distribution of trips originating on residential land in the Washington, D. C., survey area on an average weekday in 1955 [1]

Income groups [2]		Percentage distribution of trips from—							All "from" trips	
Code	Amount	Work	Personal business	Serving passengers	Changing mode of travel	Social and recreational	Medical, dental, and eating	Home	Number	Percent
	Dollars									
2	2,500-4,499	2.5	----	----	----	7.5	----	90.0	2,040	100.0
3	4,500-6,999	4.6	1.7	4.5	0.5	4.5	0.6	83.6	39,560	100.0
4	7,000-9,999	4.5	2.7	4.0	.8	5.5	.6	81.9	43,671	100.0
5	10,000 and over	7.4	1.9	6.3	.8	6.1	1.1	76.4	28,848	100.0
All groups		5.2	2.1	4.7	.7	5.4	.7	81.2	114,119	100.0

[1] Based on study of 118 selected residential subzones.
[2] The assignment of an income group rating to each area was made by the National Capital Regional Planning Council. The assignment was based on an analysis of average (median) family incomes which were developed for each area with the aid of local planning commissions, using 1950 census data and income statistics reported by the Washington Evening Star Consumer Survey of 1955–56.

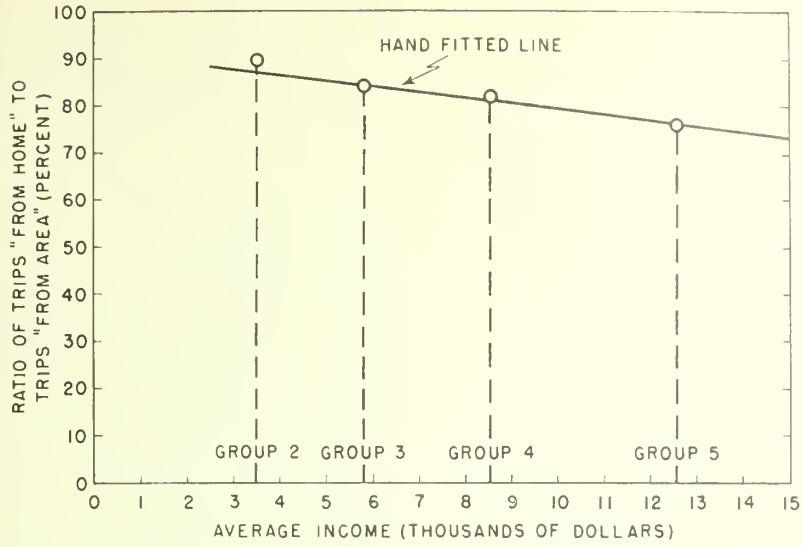

Figure 1.—Average income related to proportion of trip origins from home in 1955.

business district increased. The higher percentages of from home trips shown for rings 2, 3, and 10, compared with the other rings, may not be significant because, as shown in table 3, there were fewer interviews in these rings. This variation from the pattern is reflected in the instability of trip purpose.

Effect of variations in family income

As distance from the central business district was not an influencing factor in the proportion of residential area trips having home as a trip purpose, an analysis was made of the effect of variation in family incomes on these home-oriented trips. The results of grouping the 118 residential subzones into 5 categories of income are shown in table 5. The data indicate that the percentage of total trips originating on residential land which were from home decreased as family income increased. Although a variety of inferences could be drawn from the data, including the apparent and reasonable increase in work opportunities at homes in the highest income group areas, there are several factors left unexplained by the data. A further classification of annual income exceeding $10,000 might help to clarify the trend.

To obtain factors by which total trips originating in residential areas could be estimated, the proportion of trips from home to the total trips originating in the 118 residential subzones was plotted in relation to income, as shown in figure 1. By smoothing out the irregularities in the plotted data with a hand-fitted line, the percentages of from home trips for each income group were slightly modified, and expansion factors were developed as shown in table 6.

Table 6.—Factors developed for estimating total trips, by income groups, originating in residential areas

Income groups		Percentage of "from home" trips to total trips from area	Trip expansion factors [1]
Code	Amount (dollars)		
2	2,500–4,499	87.2	1.15
3	4,500–6,999	84.5	1.18
4	7,000–9,999	81.2	1.23
5	10,000 and over	76.4	1.31
Average, all groups		81.2	1.23

[1] 100.0 divided by percentages given in column 3.

Prior to testing the adequacy of these trip expansion factors to duplicate total trips originating in the residential areas (from all purposes), those subzones which had less than 1,000 trip origins were eliminated, or where possible combined, to provide greater stability. This resulted in a total of 48 purely residential areas composed of 77 of the original 118 subzones, either individually or in combination, being used to test the expansion factors.

The from home trips for each of the 48 areas were multiplied by (1) the applicable income-group expansion factor and (2) by the average factor of 1.23, derived from the finding that an average of 81.2 percent of trip origins in residential areas were from home. The results of these expansions compared with actual trip origins are shown in table 7.

From a detailed study of data presented in table 7, it appears that factoring by income groups improves slightly the accuracy of estimating trip origins over using an average factor. Two-thirds of the estimates were within ±8 percent of the actual values when using income factors, as compared with ±9 percent when using the average factor. The maximum error was +19 percent when factored by income, and −21 percent when using

Table 7.—Comparison of the number of trips originating in 48 residential areas in the Washington, D. C., survey area on an average weekday in 1955 with the number estimated by the application of factors to the "from home" trips

Subzones	Income group [1]	"From home" trips	Total trips originating in area	"From home" trips times income-group factor [2]	Percentage difference	"From home" trips times average factor [3]	Percentage difference
4144	2	1,161	1,334	1,335	----	1,428	7
1712-3	3	1,238	1,787	1,461	−18	1,522	−15
3143	3	2,076	2,338	2,419	5	2,553	9
3312	3	931	1,069	1,098	3	1,145	7
4752	3	1,586	1,830	1,871	2	1,950	7
4861	3	1,238	1,427	1,461	2	1,522	7
5143	3	1,852	2,006	2,185	9	2,278	14
5314	3	1,795	2,393	2,118	−11	2,208	−8
5325	3	1,089	1,395	1,285	−8	1,339	−4
5653	3	1,389	1,581	1,639	4	1,708	8
5741	3	1,201	1,218	1,417	8	1,477	12
6353	3	1,780	2,107	2,100	----	2,189	4
6469	3	1,790	2,097	2,112	1	2,202	5
6611-2-3	3	3,017	3,475	3,560	2	3,711	7
7825	3	1,101	1,244	1,299	4	1,354	9
7872-1	3	1,359	1,638	1,615	−1	1,683	3
8732	3	2,497	3,106	2,946	−5	3,071	−1
1331	4	1,102	1,195	1,355	13	1,355	13
1622-3	4	2,774	3,143	3,412	8	3,412	8
2245	4	1,079	1,117	1,327	19	1,327	19
2325	4	785	1,022	965	−6	965	−6
2333-4	4	1,010	1,365	1,279	−6	1,279	−6
2622-3	4	2,473	2,955	3,042	19	3,042	19
2643	4	890	1,055	1,094	4	1,094	4
2872	4	1,431	1,730	1,760	2	1,760	2
3434-6-7	4	1,547	1,832	1,902	4	1,902	4
3444	4	1,217	1,493	1,497	----	1,497	----
3524-6	4	1,890	2,325	2,325	----	2,325	----
3581-2	4	1,196	1,736	1,471	−15	1,471	−15
6476	4	1,194	1,487	1,468	−1	1,468	−1
7311-3	4	2,649	3,111	3,258	5	3,258	5
7452	4	1,265	1,403	1,556	11	1,556	11
7842-6-7	4	2,784	3,614	3,424	−5	3,424	−5
7911	4	784	1,098	964	−12	964	−12
8242-3	4	3,605	4,277	4,434	4	4,434	4
8426	4	913	1,247	1,122	−10	1,122	−10
1164	5	1,736	2,690	2,266	−16	2,127	−21
1322-3-5	5	1,737	2,146	2,275	6	2,136	1
2341-2-3	5	1,578	2,280	2,067	−9	1,941	−15
2421-5-7	5	1,822	2,551	2,387	−6	2,241	−12
2512	5	1,248	1,736	1,634	−6	1,535	−12
2521-2-3-5-6-7-8	5	3,077	3,900	4,031	3	3,785	−3
2531-5-6	5	1,997	2,465	2,616	6	2,456	----
2613	5	1,019	1,136	1,334	17	1,253	10
2732	5	714	1,098	935	−15	878	−20
2743	5	1,597	2,128	2,092	−2	1,964	−8
2851	5	1,241	1,411	1,625	15	1,526	8
8343	5	984	1,431	1,289	−10	1,210	−15

[1] See tables 5 and 6.
[2] Factors used to expand home trips to total trips for each income group are given in table 6.
[3] An average factor of 1.23 was used to expand home trips to total trips.

Table 8.—Residential characteristics of 48 selected areas in the Washington, D. C., survey area on an average weekday in 1955

Subzones	Number of automobiles per 100 residents	Number of residents per residential acre	Trips from home divided by trips from area	Trips to and from home divided by total trips by residents	Number of trips per resident
			Percent	*Percent*	
3434-6-7	14	81	84	92	1.2
3312	17	86	87	93	1.8
4144	20	184	87	94	1.0
5143	21	144	92	95	1.3
6353	23	97	84	84	1.5
3444	26	92	82	95	1.5
2421-5-7	27	21	71	78	1.9
5314	27	73	75	80	1.6
5325	27	52	78	91	1.5
5741	27	26	91	81	1.9
2512	29	49	72	76	2.5
5653	29	24	88	86	1.5
2245	30	45	97	82	2.5
6611-2-3	30	30	87	80	1.7
7825	30	61	88	93	1.8
8242-3	30	23	84	72	2.6
3143	31	286	89	81	2.2
4752	31	32	87	83	2.3
8426	31	13	73	81	2.9
3524-6	32	77	81	79	1.6
7872-4	32	11	84	82	2.2
8732	32	5	80	79	2.3
4861	33	50	87	88	1.6
6476	33	64	80	68	2.4
7311-3	33	28	85	55	2.7
1164	34	38	64	82	2.0
1622-3	34	14	88	82	2.5
2613	34	16	90	82	2.3
2872	34	12	83	71	1.9
0469	34	57	85	69	2.5
7452	34	64	90	94	1.8
2743	35	18	75	80	2.8
1712-3	36	12	69	76	2.8
2341-2 3	36	27	69	77	2.0
1331	37	29	92	68	2.8
2333-4	37	31	76	73	2.6
2643	37	31	84	78	2.2
2851	37	5	88	74	2.5
3581-2	37	29	69	80	2.0
8343	41	21	69	80	2.4
2521-2-3-5-6-7-8	42	20	79	74	3.2
2531-5-6	42	33	81	82	2.6
7842-6-7	42	17	77	89	2.1
7911	42	8	71	76	2.2
2325	45	45	77	88	2.9
1322-3-5	46	10	81	78	2.7
2622-3	46	65	84	80	2.4
2732	49	14	65	70	2.7

an overall expansion factor. In appraising these results, it should be remembered that the comparisons were made for individual subzones, and that the basic trip data for any one subzone could have a sizeable error. Additional study shows that the error decreases as trip volumes increase, when using either income factors or the average factor to expand from home trips.

From the foregoing the conclusion may be drawn that if the number of from home trips of any residential area is known, the total trips generated by that area can be estimated with an acceptable degree of reliability. Thus, trips generated by the residential land in any zone can be assigned to that land.

Automobile ownership and population density

Now that trips can be assigned to residential land, it should be possible to relate these known values to the various characteristics of the residential area. It has been shown that the proportion of trips originating in residential areas that are from home tends to decrease as income increases. There are, however, other influencing factors which must be considered. To determine the effect that automobile ownership and population density had on the percentage of from home trips, correlations between each of the independent variables and the percentage of from home trips were made for these same residential areas. The results of these analyses are plotted in figures 2 and 3. (Residential characteristics and trip data for these areas are presented in table 8.) Regression equations were fitted to the points by the method of least squares. Although these points do not indicate a marked trend, especially those in figure 3, related studies have shown a measurable relation between trips and both automobile ownership and population density. It is believed that the sampling variability may have somewhat obscured the relations in this case, but they should not be ignored.

Since, for a given area, the trips to home are generally about equal to the trips from home, then 81.2 percent of all trips to and from the selected residential areas are home-oriented. In effect, this means that on an average 81.2 percent of all trips to and from residential areas in the Washington area are made by the residents of those areas. The percentage varies with income, automobile ownership, and population density.

In summary, the total number of trip-ends in any residential area can be determined, if the number of trips to home and from home are known, by expanding the home-trip volumes by income, automobile-ownership, or population-density factors. An attempt will now be made to develop methods for estimating trip-ends on residential land from a knowledge of total residents trips to all origins and destinations.

Residents Trips Between All Origins and Destinations

An analysis of the data developed by the Washington transportation studies of 1948 and 1955 showed that 84 percent of all residents internal area trips began or ended at home in each of the 2 years studied.[5] The percentages of residents trips which were made to and from home were computed for 48 selected purely residential areas and are presented in table 8. Table 9 shows the percentages for 118 subzones grouped by income and distance from the central business district. Correlations were made between these percentages and the previously mentioned independent variables. The results of these analyses are presented in figures 4–6, and show

[5] A study of travel patterns in 50 cities showed that an average of 82 percent of the total trips by residents had either an origin or destination at home. The range was from about 70 to 92 percent.

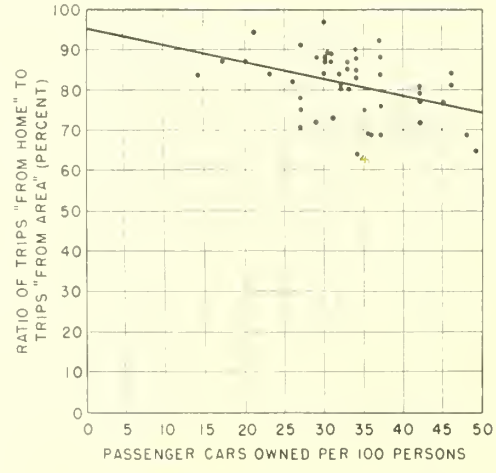

Figure 2.—Automobile ownership related to proportion of trip origins from home in 1955.

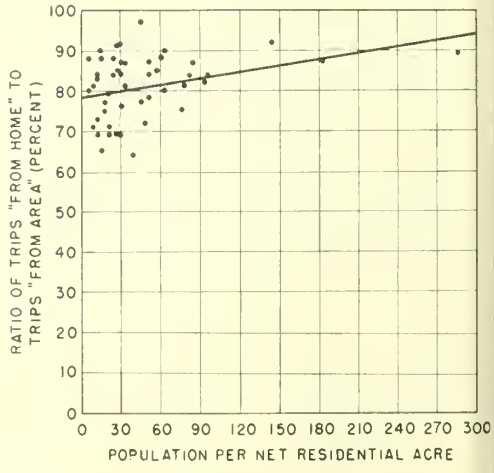

Figure 3.—Population density related to proportion of trip origins from home in 1955.

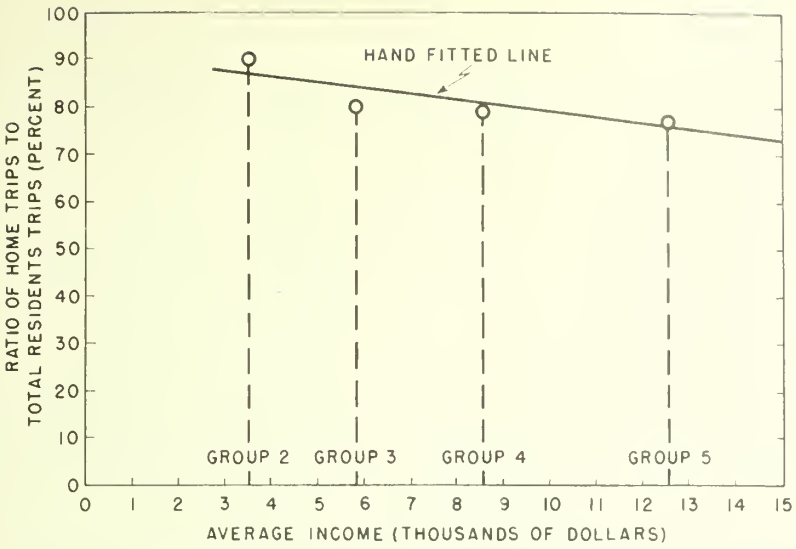

Figure 4.—Average income related to proportion of residents trips to and from home in 1955.

that the percentage of residents trips that started or ended at home tends to decrease as either income or automobile ownership increases, but appears to increase as population density increases. These patterns, it should be noted, have approximately the same relation that the percentage of from home trips to total trips originating in a residential area had to the variables.

It may be concluded, therefore, that the ratio of home trips to total residents trips is related to the ratio of home trips to total trip-ends. It follows then that the total number of trips by the residents of an area should be a useful index for estimating the total number of trip-ends attributable to the residential land use in that area, by both residents and nonresidents.

The factors to convert residents trips to residential land-use trip-ends were developed for each of the selected independent variables and are presented in table 10. As these conversion factors do not vary appreciably with any of the independent variables, a conversion factor of 1.0 was used. As previously indicated, this means that the total number of trip-ends in a residential area is equal to the total number of trips made by the residents of the area between all origins and destinations.

The number of trip-ends were then estimated from residents trips, using the factor of 1.0, and compared with the actual number of trip-ends, as determined from the transportation survey, in 35 purely residential areas which had 2,000 or more trip-ends and in which residents of 20 or more sample dwelling units were interviewed. These data are presented in table 11. This comparison shows that in 3 out of 4 cases, the estimated number of trip-ends was within 10 percent of the survey results. Part of the error that does exist is undoubtedly due to the small size of the areas studied, and to the use of only one conversion factor.

Thus, the total number of trip-ends attributable to the residential land use of an area can be estimated with an acceptable degree of accuracy from the total number of trips made by the residents of that area. The problem now is how best to determine residents trips.

Residents Trip Production

A previous analysis based on travel data obtained from the 1948 Washington, D. C., metropolitan transportation study determined the influencing effect of each of four variables, individually and combined, on the number of trips made by the residents of 95 census tracts in the District of Columbia.[6] These variables were distance from the central business district, income, automobile ownership, and population density. A technique known as "analysis or partition of the variance" was employed to estimate the significance of each of the independent variables. It was established in this study that the use of all four variables combined did not significantly increase the accuracy of predicting trips over that which was obtained using only automobile ownership and population density combined. Furthermore, automobile ownership was found to be the most reliable single predictor, with very little additional accuracy gained by combining it with population density.

[6] See footnote 2, page 92.

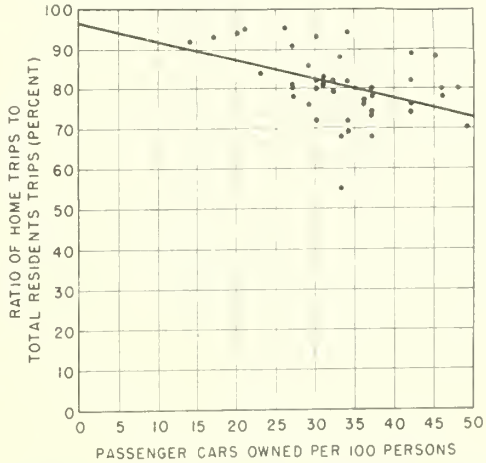

Figure 5.—Automobile ownership related to proportion of residents trips to and from home in 1955.

Table 9.—Effect of income and distance from the central business district on the percentage of residents trips to and from home on an average weekday in 1955 [1]

Variables	Number of residents trips to and from home	Total trips by residents	Percentage of to and from home trips to total trips
Income groups: [2]			
2	3,670	4,071	90.1
3	66,138	82,212	80.4
4	71,558	90,280	79.3
5	44,082	57,306	76.9
Rings: [3]			
2	6,474	7,574	85.5
3	6,334	6,853	92.4
4	26,398	32,988	80.0
5	44,898	56,973	78.8
6	31,794	39,636	80.2
7	30,492	39,481	77.2
8	18,662	23,386	79.8
9	16,022	20,793	77.1
10	4,374	6,185	70.7
Total	185,448	233,869	79.3

[1] Based on a study of 118 selected residential subzones.
[2] See tables 5 and 6.
[3] Distance (miles) of residence from the central business district. None of the selected subzones were in rings 1, 11, 12, or 13.

Assuming that automobile ownership remained the most reliable single indicator of residents trips, the 1955 data on trips per person were correlated with automobiles owned per 100 persons to determine an estimating equation for residents total trips. Total residents trip production and information concerning automobile ownership are obtainable for any area where a home-interview traffic study has been made and not just those areas which are purely residential. This correlation was based on 200 areas or zones for which the required information was available from both the 1948 and 1955 Washington surveys. The estimating regression equation is $Y=0.6+0.04X$, where Y equals trips per person, and X equals passenger cars owned per 100 persons. The correlation coefficient is $+0.71$ and the standard error of estimate is 0.39 trip per person.

Using this equation and the conversion factor of 1.0 developed previously, the residents trips and, consequently, the resi-

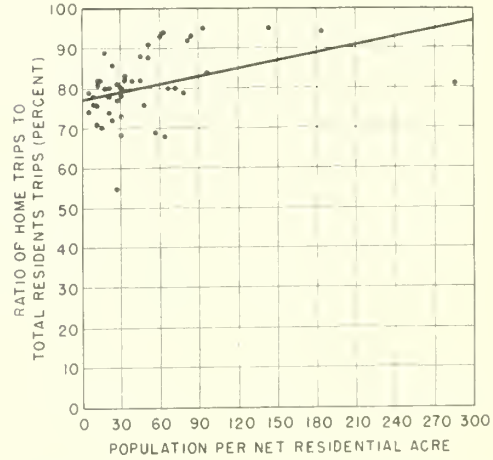

Figure 6.—Population density related to proportion of residents trips to and from home in 1955.

Table 10.—Conversion factors for estimating number of trip-ends in residential areas in the Washington, D. C., survey area on an average weekday in 1955

Independent variables	Trip-ends at home related to total trip-ends in a residential area	Residents trips to and from home related to total trips	Total residents trips related to total trip-ends in a residential area (col. 3 divided by col. 2)
Rings: [1]	*Percent*	*Percent*	
2	88.1	85.5	0.97
3	87.7	92.4	1.05
4	79.5	80.0	1.01
5	80.0	78.8	.98
6	80.8	80.2	.99
7	80.9	77.2	.95
8	82.8	79.8	.96
9	81.6	77.1	.94
10	85.8	70.7	.82
Income groups: [2]			
2	90.0	90.1	1.00
3	83.6	80.4	.96
4	81.9	79.3	.97
5	76.4	76.9	1.01
Automobile ownership [3]			
5	92.6	93.7	1.01
10	90.6	91.4	1.01
15	88.6	89.1	1.01
20	86.5	86.7	1.00
25	84.5	84.4	1.00
30	82.5	82.1	1.00
35	80.4	79.8	.99
40	78.4	77.4	.99
45	76.4	75.1	.98
50	74.4	72.8	.98
Population density [4]			
30	80.3	79.5	.99
60	81.8	81.4	1.00
90	83.3	83.3	1.00
120	84.7	85.2	1.01
150	86.2	87.1	1.01
180	87.7	89.0	1.01
210	89.9	90.9	1.02
240	90.6	92.8	1.02
270	92.1	94.7	1.03
300	93.6	96.5	1.03

[1] Distance (miles) of residence from the central business district. None of the selected subzones were in rings 1, 11, 12, or 13.
[2] See tables 5 and 6.
[3] Automobiles owned per 100 residents.
[4] Persons per residential acre.

Table 11.—Comparison of estimated and actual trip-ends in 35 residential areas in the Washington, D. C., survey area on an average weekday in 1955

Subzones	Distance from the central business district	Number of home interviews	Estimated number of trip-ends [1]	Actual number of trip-ends	Percentage difference, estimated and actual trip-ends
	Miles				
3134	2	29	5,106	4,676	9.2
5143	3	24	3,903	4,012	−2.7
1164	4	31	4,220	5,380	−21.6
2421-5-7	4	25	4,650	5,102	−8.8
3434-6-7	4	24	3,359	3,664	−8.3
5314	4	30	4,468	4,786	−6.6
7311-3	4	124	6,365	6,222	2.3
3524-6	5	26	4,779	4,650	2.8
6353	5	20	4,234	4,214	.5
6469	5	21	5,174	4,194	23.4
7825	5	55	2,367	2,488	−4.9
7842-6-7	5	96	6,248	7,228	−13.6
2512	6	20	3,297	3,472	−5.0
2521-2-3-5-6-7-8	6	25	8,300	7,800	6.4
2613	6	30	2,485	2,272	9.4
6611-2-3	6	107	7,578	6,950	9.0
7452	6	47	2,689	2,806	−4.2
8343	6	24	2,460	2,862	−14.0
1622-3	7	60	6,770	6,286	7.7
2622-3	7	104	6,177	5,910	4.5
2732	7	21	2,037	2,196	−7.2
2743	7	39	3,996	4,256	−6.1
7872-4	7	37	3,350	3,276	2.2
8242-3	7	86	10,024	8,554	17.2
8426	7	26	2,244	2,494	−10.0
1712-3	8	31	3,253	3,574	−9.0
2643	8	32	2,290	2,110	8.5
4752	8	37	3,807	3,660	4.0
4861	8	45	2,824	2,854	−1.0
5741	8	32	2,965	2,636	12.5
2851	9	33	3,355	2,822	18.9
5653	9	48	3,242	3,162	2.5
7911	9	22	2,060	2,196	−6.2
8732	9	67	6,347	6,212	2.2
2872	10	48	4,041	3,460	16.8

[1] Estimated trip-ends equals residents trips.

dential trip-ends were estimated for each of the 35 residential areas listed in table 11. The estimated trip-ends were compared with the actual survey values to test the accuracy for prediction purposes. Twenty-five or 71 percent of the estimates were within ±15 percent of the actual survey values, and 88 percent were within ±25 percent. The average error was 14.5 percent.

An intensive study of those areas with an extremely high percentage of error might result in a reasonable explanation of the differences. Once again, it should be remembered that these comparisons refer to relatively small areas and the so-called actual number of residents trips is subject to error due to sample variability inherent in the basic survey.

Therefore, it is also possible to estimate with a fair degree of accuracy the trip-ends of the residential portion of an area if the population and automobile ownership for that area are known. It must be pointed out, however, that these estimates are based upon a particular overall citywide relationship between residents trips per person and automobile ownership.

Residential Trips As a Measure of Nonresidential Trip-Ends

If, as has been shown, the number of residents trips provides a useful basis for estimating total trip-ends in residential areas, it is reasonable to test its application as a measure of nonresidential trip-ends. Obviously, the difference between total trip-ends in an urban area and trip-ends on residential land could be assigned to nonresidential land. But, it has not been practicable to isolate each parcel of residential land to determine the trips generated thereby.

However, in this study the total number of home trips is known, and these must have been generated by residential land. It can be assumed then, that these home trips represent the same proportion of total trips to all residential land as the home trips in the 118 selected (purely residential) subzones are to the total trips destined to these areas. In this manner, it was found that 19 percent of the trips destined to residential land wer

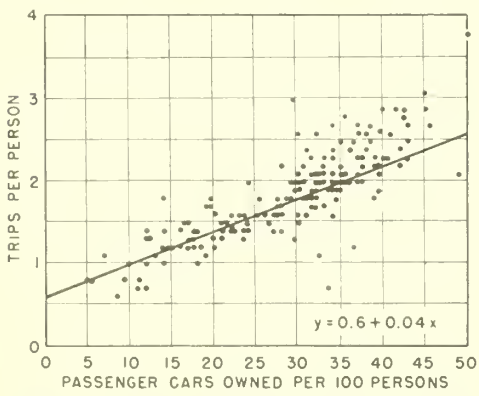

Figure 7.—Trips per person related to automobile ownership in 1955.

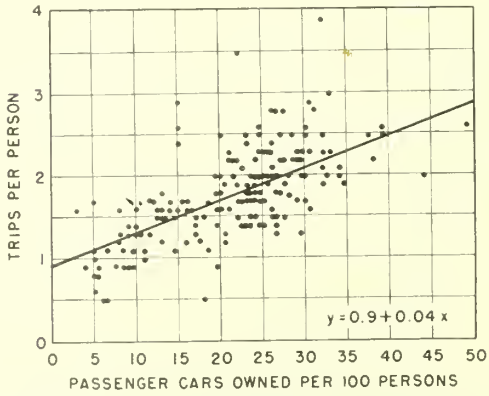

Figure 8.—Trips per person related to automobile ownership in 1948.

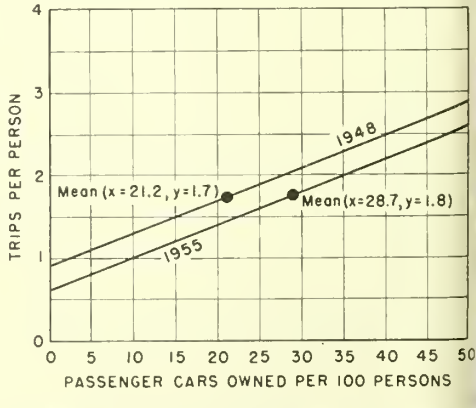

Figure 9.—Comparison of relations between trips per person and automobile ownership in 1948 and 1955.

Table 12.—Distribution of internal trip-ends by land use

Land use	Washington, D. C., surveys		Detroit, Mich.	50 cities
	1948	1955		
Residential:	*Pct.*	*Pct.*	*Pct.*	*Pct.*
Home	42	42	39	41
Other	11	11	14	11
Nonresidential	47	47	47	48
Total	100	100	100	100

other than to home. Also, since 58 percent of all trips in the metropolitan area were for other than to home purposes, it follows that 19 percent of 58 or 11 percent of all trips are generated by residential land for a purpose other than to home. This means a total of 53 percent (42+11) of all trip-ends are on residential land and the remaining 47 percent are on nonresidential land. In table 12, these results are compared with the Washington, D. C, 1948 data and with those for 50 cities, similarly developed, and with data from Detroit, Mich.

Stability of Residents Trips Per Person and Automobile Ownership

To determine the stability of the relation between residents trips per person and automobile ownership, an additional correlation between the two variables was made for the same 200 areas using data developed from the 1948 Washington area transportation survey. The results of this analysis are presented in figure 8. A comparison of the regression line in figure 8 and the regression line based on 1955 data (fig. 7) is shown in figure 9. A study of these two lines indicates that the relation between trips and automobile ownership has not been stable, but has shifted measurably during the 7-year period. The only difference between the two lines is the value of the constant (0.9 for 1948 and 0.6 for 1955) where the regression line intersects the Y axis. Statistical t tests showed that

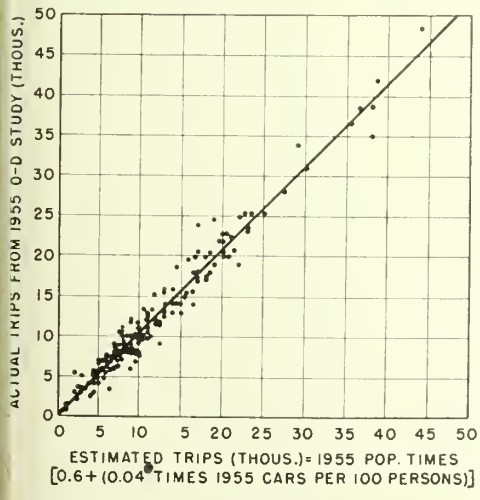

Figure 10.—Actual trips related to estimated trips. Estimates based on relation of trips per person and automobile ownership.

Table 13.—Indices for estimating the number of trips to and from residential subdivisions of the Washington, D. C., survey area [1]

Automobile ownership per 100 residents in each subdivision	Trips per resident, based on automobile ownership (per 100 residents) values for the entire survey area of—										
	20	21	22	23	24	25	26	27	28	29	30
5	1.20	1.16	1.12	1.08	1.04	1.00	0.96	0.92	0.88	0.84	0.80
10	1.40	1.36	1.32	1.28	1.24	1.20	1.16	1.12	1.08	1.04	1.00
15	1.60	1.56	1.52	1.48	1.44	1.40	1.36	1.32	1.28	1.24	1.20
20	1.80	1.76	1.72	1.68	1.64	1.60	1.56	1.52	1.48	1.44	1.40
25	2.00	1.96	1.92	1.88	1.84	1.80	1.76	1.72	1.68	1.64	1.60
30	2.20	2.16	2.12	2.08	2.04	2.00	1.96	1.92	1.88	1.84	1.80
35	2.40	2.36	2.32	2.28	2.24	2.20	2.16	2.12	2.08	2.04	2.00
40	2.60	2.56	2.52	2.48	2.44	2.40	2.36	2.32	2.28	2.24	2.20
45	2.80	2.76	2.72	2.68	2.64	2.60	2.56	2.52	2.48	2.44	2.40
50	3.00	2.96	2.92	2.88	2.84	2.80	2.76	2.72	2.68	2.64	2.60

[1] Determined from correlation of trips per resident and automobile ownership in the Washington, D. C., survey area, 1948 and 1955. To compute number of trips, multiply population of each subdivision by appropriate index.

the difference between these two constants was highly significant and could not be accounted for by sampling variability. Measures obtained from the analyses are as follows:

	1948	*1955*
Correlation coefficient	+0.67	+0.71
Standard error of estimate	.41	.39
Average trips per person, $\bar{y}$	1.7	1.8
Average automobile ownership per 100 persons, $\bar{v}$	21.2	28.7

Further analysis of the regression lines gives an insight into the reasons for the shift. The fact that the slope of the regression lines (0.04) remained the same during the interval between the study periods is of particular importance. Equally important is the fact that, although the average automobile ownership increased from 21.2 to 28.7 automobiles per 100 persons, an increase of 35 percent, the average number of trips per person remained relatively constant. The explanation, therefore, for the shift in the regression lines appears to be the increase in automobile ownership. This, in effect, means that during the 7-year interval the numerical relationship between the two variables changed due to the increase in automobile ownership in 1955, but the relative association between the two variables, as indicated by the correlation coefficients, remained almost constant.

If these relations are valid and the trend is assumed to be consistent in the future, then it should be possible to forecast residents trip production and residential land-use trip generation in the Washington area for any future year, provided that population and automobile ownership for that year are known or can be accurately forecasted. In fact, estimates could be readily obtained from the indices developed in table 13, which may be taken from a family of regression curves assuming a constant average number of trips per person and a uniform slope, similar to those in figure 9. The use of this table would require estimates of the citywide average automobile ownership for the future year plus estimates of automobile ownership for each zone or area for which potential trip data are desired.

Residents trip production for 1955 for each of the previously mentioned 200 areas was estimated by the indices in table 13. The estimated trips were correlated with the actual trips as found in the survey, and the data are presented in figure 10. It is readily apparent that the degree of association between the estimated and actual trips is very high. The square of the correlation coefficient (+0.98)

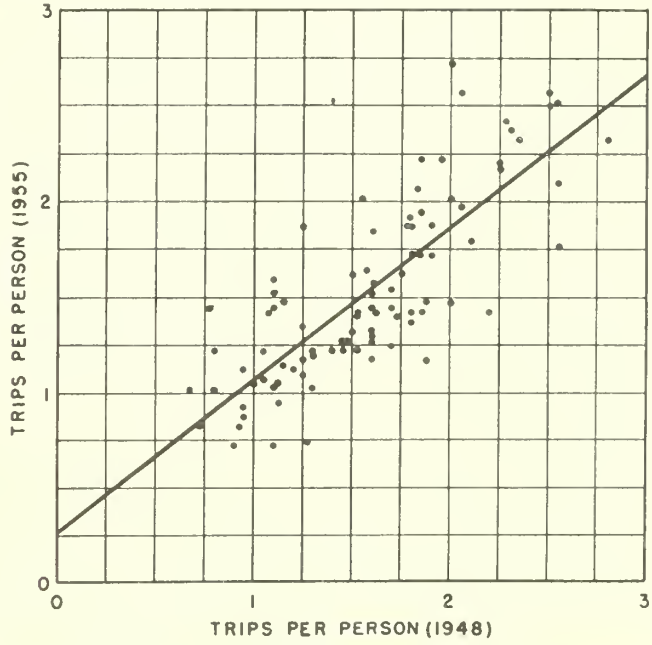

Figure 11.—Relation of trips per person in 1948 and 1955.

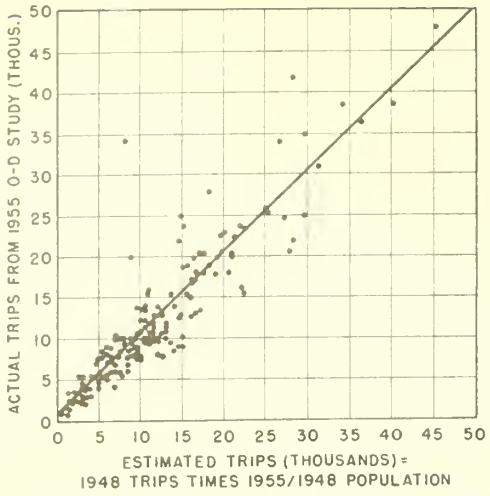

Figure 12.—Actual trips related to estimated trips. Estimates based on change in population between 1948 and 1955.

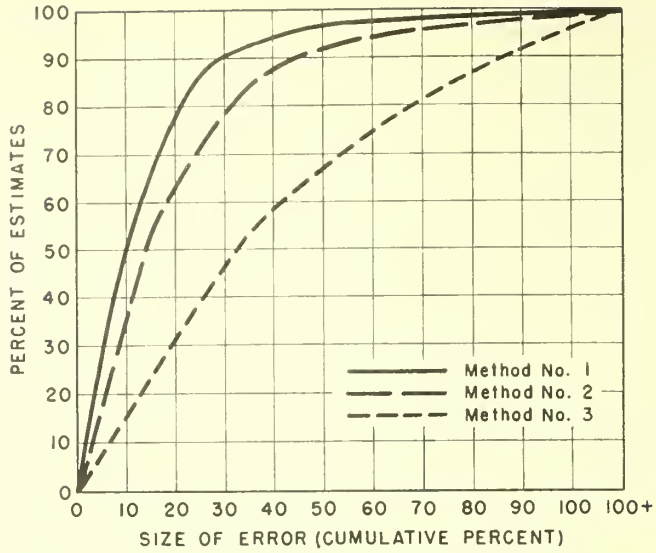

Figure 13.—Comparison of three methods for estimating residents trips.

indicates that the change in automobile ownership and change in population over the 7-year period explained 96 percent of the variation in trip production (residents trips) and residential land trip generation for 1955. Two-thirds of the estimated values were within ±15 percent of the survey results.

In the event automobile ownership data are not available or are too difficult to develop, one or more of the other previously mentioned independent variables could be substituted with, of course, a probable decrease in predictability.[7]

Stability of Residents Daily Trip Production

To forecast accurately residents trip production and residential area trip generation by the method just described would require accurate estimates of population, as well as one or more factors related to trip production and generation, such as automobile ownership, income, etc., for each zone or area for which trip forecasts are desired. Admittedly, this process could prove to be more difficult, and the results perhaps would not be as satisfactory as those obtained from estimates derived from only one independent variable.

The previous comparison between 1948 and 1955 data showed that average trips per person remained almost constant during the 7-year period. Assuming that this consistency is not just a happy balance of plus and minus values, but rather the result of a consistency in the individual figures which comprise this average, then a second and easier method for projecting trip production and residential land trip generation to future years will be available.

To test this consistency or stability of residents daily trip production, trips per person in 1948 were compared with trips per person in 1955 for 95 census tracts in the District of Columbia. The results of this comparison are shown in figure 11. The correlation coefficient is +0.80. This means that the same factors that affected trips in 1948 were still applicable in 1955. Therefore, it can tentatively be assumed that residents trip production in any desired area for which prior survey results are available can be forecasted by multiplying trips per person for the base year by population of the area for any future year.

Residents trips for 1955 for each of the previously mentioned 200 areas were estimated in this manner, using 1948 trip data as the base. A comparison between the estimated and actual survey values is shown in figure 12. The results of this analysis show that two-thirds of the estimated values were within ±25 percent of the actual values (see method 2 in table 14) and the correlation coefficient was +0.94. In other words, this means that the change in population alone was 88 percent effective in explaining the 1955 residents trip production and, consequently, residential land-use trip generation.

Growth in Automobile Ownership as an Indicator of Potential Trips

To measure the effect of automobiles owned, alone, as a predicting variable for estimating 1955 residents trip production and residential land-use trip generation, 1948 trips for the 200 areas were multiplied by the ratio of 1955 automobiles owned to 1948 automobiles owned. The analysis revealed that this method of estimating residents trips was not nearly as effective as the two previous methods described. In this case the correlation coefficient was +0.88. Data in table 14 and figure 13 compare the accuracy of three methods developed for estimating 1955 residents trips and residential trip generation for the 200 areas studied.

Estimating Future Traffic Potential of Residential Areas

Assuming that the travel patterns of the Washington, D. C., metropolitan area are not unlike the travel patterns of other cities, the following methods and procedures have been developed for estimating with a fair degree of accuracy total residents trips for any area. As the factor to convert residents trips to trip-ends on residential land was found to be approximately equal to 1.0, these methods will also give the trip generation due to any residential portion of the study area.

Method No. 1

1. Compute the residents trips per person for each zone in the area for which population and trip data are given.

2. Compute automobile ownership (per 100 persons) for each zone.

3. Let x=automobiles per 100 persons, y=trips per person, and n=number of zones. Solve the following simultaneous equations for a and b:

$$\Sigma y = an + b\Sigma x$$

$$\Sigma xy = a\Sigma x + b\Sigma x^2$$

Table 14.—Comparison of three methods for estimating 1955 residents trips for 200 areas in the Washington, D. C., survey area

Maximum error in percent (plus or minus)	Estimating method No. 1 [1]	Estimating method No. 2 [2]	Estimating method No. 3 [3]
	Percent of estimates	*Percent of estimates*	*Percent of estimates*
5	30.9	18.5	7.8
10	51.0	39.5	15.7
15	67.2	54.5	26.2
20	79.4	63.5	33.0
25	88.7	68.5	40.3
30	91.2	78.0	47.6
35	92.2	86.0	53.9
40	94.1	88.0	57.6
45	95.6	91.0	63.9
50	96.6	93.5	68.6
55	97.1	95.0	71.2
60	97.1	95.0	75.4
65	97.1	96.5	79.1
70	98.0	97.0	82.2
75	98.0	98.5	85.9
80	98.5	98.5	87.4
85	98.5	98.5	88.0
90	98.5	98.5	89.0
95	98.5	99.0	89.5
100	98.5	99.0	92.1
Average error	±15.4	±20.9	±50.3

[1] Estimating equation: y=1955 population [0.6+(0.04× 1955 automobiles per 100 residents)]

[2] Estimating equation: y=1948 residents trips × 1955 population/1948 population.

[3] Estimating equation: y=1948 residents trips × 1955 automobile ownership/1948 automobile ownership.

[7] See footnote 2, page 92.

4. Determine the estimating equation ($y=a+bx$). This equation provides an estimate of the trips per person corresponding to automobile ownership in a particular area for an overall citywide automobile ownership value ($\Sigma x/n$) existing at the time of the survey. To apply this relationship to a future time, it is necessary to compute the parallel curve for an overall automobile ownership value estimated for the future data ($\Sigma x'/n$). This is done by assuming that y and b remain constant and by solving for the new a in the following equation: $\bar{y}=a'+b\bar{x}$.

The trips per person for each area can then be calculated by substituting in the new equation ($y=a'+bx'$) for each estimated value of automobile ownership in the study areas. If desired, an index table similar to table 13 could be prepared.

Although this has not been tested, instead of using automobile ownership data, it should be possible to utilize the techniques just described by substituting population density, income, distance from the central business district, or a combination of these variables.

Method No. 2

1. Compute the ratio of residents trips per person for each zone in the area.
2. Estimate the future population of each zone.
3. Multiply 1 times 2 for each respective zone.
4. In zones for which prior trip data are not available, estimates can be made by comparison with zones having similar characteristics.

Method No. 3

1. Determine the number of trips and automobiles owned for each zone in the area.
2. Estimate the future number of automobiles owned in each zone.
3. Multiply the number of trips by the ratio of future to present automobiles owned.

The application of the most desirable method of estimating the potential trip generation in residential areas is dependent upon the availability and reliability of correlative data. For instance, although the first method appears to be the best, it must be pointed out that its accuracy depends upon the reliability of the estimated population and automobile ownership data. In the examples discussed in this article, the number of persons and the number of automobiles were known from origin and destination surveys. However, to estimate residential trips for a future period, the population and automobile ownership information is not available and must be estimated. Since automobile ownership estimates are likely to be less accurate than population forecasts, method No. 2, utilizing population data alone, may well be more accurate in forecasting trips.

Current Structural Bridge Steels: A Survey of Usage and Economy

BY THE BRIDGE DIVISION
BUREAU OF PUBLIC ROADS

Reported by NATHAN W. MORGAN, Bridge Engineer

THE advent of welding for steel bridges, and a demand for higher strength steels for large structures and those requiring higher working stresses, gave rise, a couple of decades ago, to a number of continuing research studies covering the characteristics of structural steels and leading to the promulgation of new specifications and the revision of others. This article summarizes and compares the characteristics of the common bridge steels available in this country and attempts to answer the more frequent questions concerning their use.

Standard bridge steels are manufactured by the leading steel producers that roll plates, shapes, and bars. The characteristics of end products are governed by the applicable standard specifications of the American Society for Testing Materials or by proprietary specifications. During the past quarter century, welding has been increasing progressively as a means of fabrication, and since the mid-century the tempo of this program has been intensified. Therefore, welding is not ignored in this survey.

ASTM, Military, and Proprietary Specifications

The standard structural steels considered here are those covered by the following ASTM Standards:

ASTM–A6–57T.—General requirements for delivery of rolled steel plates, shapes, sheet piling, and bars for structural use (*1*).[1]

ASTM–A7–56T.—Steel for bridges and buildings (*2*).

ASTM–A8.—Structural nickel steel (*3*).

ASTM–A94.—Structural silicon steel (*3*).

ASTM–A141.—Structural rivet steel (*3*).

ASTM–A195–52T.—High-strength structural rivet steel (*3*).

ASTM–A242.—High-strength low-alloy structural steel (*3*).

ASTM–A325–55T.—Quenched and tempered steel bolts and studs with suitable nuts and plain hardened washers (*3*).

ASTM–A373–56T.—Structural steel for welding (*2*).

ASTM–A406–57T.—High-strength structural alloy rivet steel (*1*).

Also considered are the following military specifications which cover high-strength steels with characteristics somewhat similar to ASTM–A94 and A242:

MIL–S–16113B (Navy), dated December 4, 1953.—Steel plate, hull and ordnance, structural, black (uncoated), and zinc-coated (galvanized).

MIL–S–20166 (Navy), dated October 3, 1951.—Steel: bars and shapes (for hull construction) (including material for drop and miscellaneous forgings).

The following proprietary structural specifications are also considered here because they are in common use:

"Medium Manganese," "Manganese Vanadium," "Mayari R," "Man-Ten A242," "Cor-Ten," "Tri-Ten," and "T1."

These steels which are produced by two companies, can be and are produced with similar or identical characteristics by other companies, but usually on order to the purchaser's specifications.

There are structural steels other than those discussed in this article. However, it is believed that their characteristics closely coincide with comparable steels discussed here, and for that reason they are omitted.

Definitions

The term "alloy" steel refers to a steel (1) in which the maximum range specified for the content of alloying elements exceeds one or more of the following limits: manganese 1.65 percent, silicon 0.60 percent, copper 0.60 percent; or (2) in which a definite range or a definite minimum quantity of any of the following elements is specified or required within the limits of the recognized commercial field of alloy steels: aluminum, boron, chromium up to 3.99 percent, cobalt, molybdenum, nickel, titanium, tungsten, vanadium, zirconium, or any other alloying element added to obtain a desired alloying effect (*4*).

The terms "low-alloy" or "high-strength" steel comprise a specific group of steels with chemical compositions specifically developed to impart higher mechanical property values and greater resistance to atmospheric corrosion than are obtainable from conventional carbon structural steels containing copper. High-strength low-alloy steel is generally produced to mechanical property requirements rather than to chemical composition limits (*5*).

In general, alloy steels and high-strength low-alloy steels are more difficult to produce than carbon steels, both in the mills and in fabricating shops. Thus, these steels are somewhat more costly. The purpose of such steels is to obtain economy in long-span or costly structures where overhead clearance is limited or where the use of higher working stresses will reduce the tonnage of steel required; and in cases where corrosion resistance is of economic importance, without increasing the thickness of metal for this purpose. These steels may or may not be of welding quality.

In the following sections of this article, the steels previously listed and the specifications governing them are discussed individually. Physical properties and prices are given in table 1; chemical analyses, in table 2. The price data are only approximate, and are given mainly for the sake of comparison. Prices vary between manufacturers and by location of mill, and are subject to frequent change; those given are as of October 1, 1957. The prices represent basic steel costs and do not include the various manufacturing extras, such as apply to special metallurgical requirements, dimensions, processing, quantity, packing, marking, and other items.

Among the steels used for highway bridges, the A7 steel remains the best choice for ordinary riveted construction. When high-strength steel is required for long-span or heavy structures, the likely choice would be either Medium Manganese or Man-Ten A242 steel.

When welding is involved, A373 steel should be used since it largely meets the metallurgical requirements for welding and forms the basis for the American Welding Society bridge specifications (*6*). When the use of high-strength steel becomes economical, the usual choice would be either Manganese-Vanadium or Tri-Ten steel. Mayari R steel may be used if welding quality is specified and the hardenable elements are thereby limited. The differential cost between A373 and A7 steels averages about $5.00 per ton except for killed steel in plates between 1 inch and 1½ inches thick, where the differential is $18.00 per ton.

For high-strength steel members, large rivets or high-strength bolts should be used. It is hoped that an ASTM standard specification governing a more satisfactory high-strength steel will soon be available.

ASTM Specification Steels

Common requirements, ASTM–A6–57T

Specification ASTM–A6 covers a group of common requirements which, unless otherwise specified in the purchase order in an

[1] Italic numbers in parentheses refer to the list of references on page 104.

ndividual specification, apply to rolled steel plates, shapes, bars, and sheet piling under each of the other ASTM standard or tentative specifications previously listed, except A325. This general specification was gathered out of the other specifications in 1949 and has been issued separately as ASTM–A6 since that time. It covers the general clauses, such as definitions, test specimens, defect, marking, inspection, permissible variations in dimensions and weights, and the welding of surface imperfections. It is therefore necessary that specification A6 be used in conjunction with each of the other specifications.

Plates are classed as flat, hot-rolled steel over 6 inches in width and 0.23 inch or over in thickness, or over 48 inches in width and 0.18 inch or over in thickness. Bars are

Table 1.—Physical properties and prices of structural steels

Steel specification	Physical properties [1]												Price of steel per pound [2]			
	Tensile strength, minimum or range		Yield point, minimum		Elongation, minimum				Reduction in area, minimum		Bend test		Plates		Other	
					In 8 inches		In 2 inches									
	Thickness, inches	P. s. i.	Thickness, inches	P. s. i.	Thickness, inches	Percent	Thickness, inches	Percent	Thickness, inches	Percent	Thickness, inches	Ratio of bend diameter to thickness	Thickness,[1] inches	Cents	Examples[3]	Cents
ASTM-A7-56T		[4] 60,000– 75,000		33,000	5/16–3/4	[5] 21	To 3 1/2	[5] 24			To 3/4	1/2	To 1 1/2	5.20	S	5.275
											3/4–1	1	1/2–1	5.20	L	5.875
											1–1 1/2	1 1/2	1–1 1/2	5.20	WF	5.625
											1 1/2–2	2 1/2	Over 1 1/2	5.95	Pl	5.85
											Over 2	3				
ASTM-A373-56T		58,000– 75,000		32,000	5/16–3/4	[5] 21	To 3 1/2	[5] 24			To 3/4	1/2	To 1 1/2	5.25	S	[6] 5.425
											3/4–1	1	1/2–1	5.45	L	6.025
											1–1 1/2	1 1/2	1–1 1/2	6.10	WF	5.975
											1 1/2–2	2 1/2	Over 1 1/2	6.20	Pl	6.10
											Over 2	3				
ASTM-A94-54		80,000– 95,000		45,000	5/16–3/4	[5] 16	To 3 1/2	[5] 19	To 3/4	[5] 30	To 3/4	1	To 1 1/2	6.45	S	6.675
											3/4–1	1 1/2	1/2–1	6.40	L	7.275
											1–1 1/4	2	1–1 1/2	6.40	WF	7.025
											Over 1 1/4	2 1/2	Over 1 1/2	6.50	Pl	7.05
Medium Manganese, similar to Man-Ten A242 [7].	To 3/4	72,000	To 3/4	50,000	To 3/4	18	To 3/4	20			To 3/4	1	To 1 1/2	6.40	S	6.525
	3/4–1 1/2	70,000	3/4–1 1/2	46,000	3/4–1 1/2	19	3/4–1 1/2	24			3/4–1	1 1/2	1/2–1	6.40	L	7.125
	1 1/2–4	65,000	1 1/2–4	42,000	1 1/2–4	19	1 1/2–4	24			1–1 1/2	2	1–1 1/2	6.40	WF	6.875
											1 1/2–2	2 1/2	Over 1 1/2	6.50	Pl	7.05
											2–4	3				
ASTM-A242-55	To 3/4	70,000	To 3/4	50,000	5/16–3/4	[5] 18	1 1/2–4	[5] 24			To 3/4	1	To 1 1/2	7.625	S	7.80
	3/4–1 1/2	67,000	3/4–1 1/2	46,000	3/4–1 1/2	[5] 19					3/4–1	1 1/2	1/2–1	7.625	L	8.40
	1 1/2–4	63,000	1 1/2–4	42,000	1 1/2–3 1/2	[5] 19					1–1 1/2	2	1–1 1/2	7.625	WF	8.15
											1 1/2–2	2 1/2	Over 1 1/2	7.725	Pl	8.275
											2–4	3				
Manganese-Vanadium and Tri-Ten (A242 steels).	To 3/4	70,000	To 3/4	50,000	To 3/4	18	To 3/4	[8] 22			To 3/4	1	To 1 1/2	7.625	S	7.75
	3/4–1 1/2	67,000	3/4–1 1/2	46,000	3/4–1 1/2	19	3/4–1 1/2	[9]			3/4–1	1 1/2	1/2–1	7.625	L	8.35
	1 1/2–4	63,000	1 1/2–4	42,000	1 1/2–4	19	1 1/2–4	24			1–1 1/2	2	1–1 1/2	7.625	WF	8.10
											1 1/2–2	2 1/2	Over 1 1/2	7.725	Pl	8.275
											2–4	3				
Mayari R (an A242 steel).	To 3/4	70,000	To 3/4	50,000	5/4 –3/4	[5] 18	To 3/4	22					To 1 1/2	7.625	S	7.80
	3/4–1 1/2	67,000	3/4–1 1/2	46,000	3/4–1 1/2	[5] 19	3/4–1 1/2	[9]					1/2–1	7.625	L	8.40
	1 1/2–4	63,000	1 1/2–4	42,000	1 1/2–4	16	1 1/2–4	[5] 24					1–1 1/2	7.625	WF	8.15
	Over 4	60,000	Over 4	40,000			Over 4	20					Over 1 1/2	7.725	Pl	8.275
Cor-Ten (an A242 steel).	To 1 1/2	70,000	To 1 1/2	50,000	To 1 1/2	18	To 1 1/2	22			To 1 1/2	1	To 1 1/2	7.625	S	7.75
	1/2–1 1/2	67,000	1/2–1 1/2	47,000	1/2–1 1/2	19	1/2–1 1/2	[9]			1/2–1 1/2	2	1/2–1	7.625	L	8.35
	1 1/2–3	63,000	1 1/2–3	43,000	1 1/2–3	19	1 1/2–3	24			1 1/2–3	3	1–1 1/2	7.625	WF	8.10
													Over 1 1/2	7.725	Pl	8.275
AREA, high-strength steel.	To 1	78,000	To 1	50,000	To 1	[10] 17	1 1/2–4	21	To 1	[10] 30	To 3/4	1	[11]	[11]	S	[11] 7.375
	1–1 1/2	75,000	1–1 1/2	47,000	1–1 1/2	[10] 18			1–1 1/2	[12] 27.5	3/4–1	1 1/2			Pl	[11] 7.90
	1 1/2–4	72,000	1 1/2–4	45,000	1 1/2–4	[10] 19			1 1/2–4	[9]	1–1 1/4	2				
											Over 1 1/4	2 1/2				
ASTM-A141-55		52,000– 62,000		28,000		24					[13]	[13]				
ASTM-A406-57T		68,000– 82,000		50,000		20										
T1	1/4–6	105,000–135,000	1/4–6	90,000			1/4–2	18	1/4–2	55	To 1	2	To 1 1/2	14.30		
							2–4	17	2–4	50	1–2	3	1/2–1	13.55		
							4–6	16	4–6	45			1–1 1/2	13.55		
													Over 1 1/2	14.30		
ASTM-A8-54		90,000–115,000		55,000		14				30						
ASTM-A195-52T		68,000– 82,000		38,000		20										
MIL-S-20166, H. T. grade.	Under 1/4	90,000	Under 1/4	50,000												
	1/4–1 1/2	87,000	1/4–1 1/2	48,000												
	1/2–1	84,000	1/2–1	45,000												
	1–2	84,000	1–2	42,000												
	Over 2	82,000	Over 2	40,000												
MIL-S-16113B, H. T. grade.	Under 1 1/2	92,000	Under 1 1/2	50,000												
	1/2–1 1/2	88,000	1/2–1 1/2	47,000												
	1 1/2–2	86,000	1 1/2–2	44,000												
	Over 2	85,000	Over 2	42,000												

[1] Thickness dimensions are exclusive of the lower limit and inclusive of the upper limit; e.g., 1 to 1 1/2 inches means over 1 inch and including 1 1/2 inches.

[2] Prices are base cost at the mill, and include mill welding of imperfections (ASTM–A6); they do not include fabrication or erection. Prices vary somewhat among manufacturers and between mills, and are subject to frequent change. The prices quoted are as of October 1957, and are for quantities of 25 tons or more.

[3] S=shapes; L=4-x4-x1/2-inch angle; WF=30-x10 1/2-inch wide flange section; and Pl=1/2- x 3/4- x 240-inch plate.

[4] For plates to 1 1/2 inches, 60,000–72,000; for all shapes and for plates over 1 1/2 inches, 60,000–75,000.

[5] For variations, see ASTM specifications (in the case of Mayari R, see ASTM–A242).

[6] Price of 5.625 cents applies to beams of the following dimensions (inches): 36 x 16 1/2, 36 x 12, 33 x 15 3/4, 33 x 11 1/2, 30 x 15, 30 x 10 1/2, 27 x 14, 24 x 14, 21 x 13, 14 x 16, 14 x 14 1/2, 12 x 12, and 10 x 10.

[7] Physical values for the two steels differ slightly.

[8] For Tri-Ten steel only.

[9] Not specified.

[10] For variations, see AREA Specification; proceedings of AREA for 1958.

[11] Check with manufacturer as to availability and price.

[12] Reduced from 30 percent by 1/2 percent for each 1/16 inch of thickness over 3/4 to 1 1/2 inches.

[13] 180 degrees bent flat cold.

199469

Table 2.—Chemical composition of structural steels (percentages by ladle analysis)

Chemical composition (percent)	Steel specifications																
	ASTM-A7-56T	ASTM-A373-56T	ASTM-A8-54	ASTM-A94-54	AREA[1]	ASTM-A141-55	ASTM-A195-52T	ASTM-A406-57T	ASTM-A325-55T	ASTM-A242-55[1]	MIL-S-20166, H. T. grade	MIL-S-16113B, H. T. grade	Medium Manganese and Man-Ten A242	Manganese-Vanadium and Tri-Ten	Mayari R[3]	Cor-Ten[3]	T1[3]
Carbon (maximum)	----	[4] 0.26–.28	0.43	0.40	0.30	----	0.30	0.08	[5] 0.30	0.22	0.18	0.18	[2] 0.27	0.22	0.12	0.12	0.10–.20
Manganese	----	[6] .50–.90	[7] .80	----	[5] 1.20	----	[7] 1.65	.25–.50	[5] .30	[7] 1.25	[7] 1.30	[7] 1.30	1.10–1.60	[7] 1.25	.50–1.00	.20–.50	.60–1.00
Phosphorus (maximum):																	
Acid	0.06	------	.04	.06	.06	0.06	.06	------	.048	----	------	------	------	----	------	------	--------
Basic	.04	.04	.04	.04	.04	.04	.04	------	.048	----	.04	.04	.04	.04	.12	.07–.15	.040
Sulfur (maximum)	.05	.05	.05	.05	.05	.05	.05	------	.058	.05	.05	.05	.05	.05	.05	.05	.050
Silicon	----	[8] .15–.30	------	[5] .20	[7] .30	----	[7] .25	.70–1.00	----	----	.15–.35	.15–.35	[7] .30	[7] .30	.10–.50	.25–.75	.15–.35
Copper	[5] [9] .20	[5] [9] .20	[5] [9] .20	[5] [9] .20	[5] [9] .20	[5] [9] .20	[5] [9] .20	.50–.75	----	----	[7] .35	[7] .35	.20–.35	[5] .20	.20–.50	.25–.55	.15–.50
Nickel	----	------	3.00–4.00	----	----	----	----	.70–1.00	----	----	[7] .25	.25	------	----	.50–1.00	[7] .65	.70–1.00
Chromium	----	------	------	----	----	----	----	------	----	----	[7] [10] .15	[7] [10] .15	------	----	.40–1.00	.30–1.25	.40–.80
Vanadium	----	------	------	----	----	----	----	------	----	----	[5] .02	[5] .02	------	[5] .02	------	------	.03–.10
Molybdenum	----	------	------	----	----	----	----	------	----	----	.05	[7] .06	------	----	------	------	.40–.60
Other [11]	----	------	------	----	----	----	----	.10–.30	----	----	[5] .005	[5] .005	------	----	------	------	.002–.006

[1] Other elements shall be added by the manufacturer to give the prescribed mechanical properties.
[2] For Medium Manganese, 0.25; for Man-Ten A242, 0.27.
[3] Typical composition.
[4] According to thickness: to ½ inch, 0.26; ½–1, 0.25; 1–2, 0.26; 2–4, 0.27; and shapes and bars, 0.28.
[5] Minimum percentage.
[6] For plates over ½ inch, group A shapes; and bars over 1 inch. For group A shapes, se sizes given in table 1, footnote 6.
[7] Maximum percentage.
[8] For plates over 1-inch thick.
[9] If specified.
[10] Chromium shall not be added intentionally, 0.00 percent chromium desirable.
[11] Specification ASTM-A406-57T, aluminum; MIL-S-20166 and MIL-S-16113B, ti tanium; and T1, boron.

classed as rounds, squares, and hexagons of all sizes, flats 1¾₄ inch (0.2031 inch) and over in thickness and not over 6 inches wide, and 0.230 inch or more in thickness and over 6 inches to and including 8 inches wide, special bar sections, and bar-size shapes under 3 inches in maximum cross-sectional dimension. Shapes are rolled, flanged sections having at least one dimension of the cross section 3 inches or greater.

Ladle analyses for each heat of steel record the percentages of carbon, manganese, phosphorus, sulfur, and other elements specified or restricted by the applicable specification. This analysis is made from a test ingot taken during the pouring of the heat. A check analysis may be made by the purchaser. The chemical composition thus determined must conform to the applicable specification for check analysis as to elements required or restricted.

Steel for bridges and buildings, ASTM–A7–56T

The ASTM–A7 specification had its origin in 1901, following the age of wrought iron. It has been revised frequenty in both physical and chemical characteristics before appearing in its present form. In 1949, the A7 specification for bridge steel was combined with the A9 specification for building steel, giving the present specification for steel in bridges and buildings.

This material is the common mild carbon steel so widely used for all kinds of steel structures, in the form of shapes, plates, and bars of structural quality. It is manufactured by both the open-hearth and electric-furnace processes. Acid Bessemer A7 steel is not used for bridges or other dynamically loaded structures.

Carbon steel meeting the A7 specification is essentially a steel for riveted construction, since welding was not in view when the specification was promulgated. There are no carbon or manganese limitations, nor is a deoxidation process of manufacture specified. This steel may be characterized as being possibly impact or notch sensitive and hardenable in the zone affected by the heat of welding, particularly for the thicker sections or under cold temperature service, and is thus not a trustworthy weldable steel. The present American Welding Society bridge specifications (*6*) permit the use of A7 steel not over 1 inch thick for minor parts not proportioned for calculated stresses.

Structural steel for welding, ASTM–A373–56T

Structural steel for welding, covered by specification ASTM–A373, has a base price somewhat greater than A7 steel.

A373 steel is recommended for use in all welded bridge construction except as otherwise stipulated by the American Welding Society bridge specifications (*6*). As has been noted, A7 steel, which is permissible for members not over 1 inch thick, may be used for minor parts not proportioned for calculated stress, or for strengthening and repair of existing structures, if it is economical to do so. This is interpreted to mean that A373 steel should be used for trusses, girders, beams, arches, rigid frames, floor beams, stringers, floor expansion devices, shoes, and other main members, but A7 steel may be used for lateral and longitudinal sway bracing, diaphragms, floor slab armor parts, railings, and similar minor parts.

For rolled beams with welded cover plates, A373 steel is required for both cover plates and beams except in cases where the engineer desires to use A7 steel, in which case approval may be given for the use of A7 steel for both beam and cover plates providing the cover plates are not over 1 inch thick. However, it is believed that there is now less reason for using A7 steel in these cases, since A373 steel has become more readily available.

A373 steel is recommended for floor expansion dams because experience has shown that the heavy impact from wheel loads tends to loosen rivets. A373 steel is recommended for welded shoes because the use of thick plates and large size fillet welds is usually more economical than steel castings. Steel H-piles to be spliced by welding may be either A373 of A7 steel.

For composite beam and girder construction the welding of shear connectors to compression flanges or to those parts of tension flanges where the tensile stress does not exceed 75 percent of the allowable stresses, A373, A7 or other steels considered weldable because of favorable conditions may be used satisfactorily. The welding of shear developers to tension flanges where the tensile stress exeeds 75 percent of the allowable stress is not considered satisfactory.

Structural nickel steel, ASTM–A8

Structural nickel steel covered by the ASTM–A8 specification is used primarily for the main stress-carrying members of large structures. The specification covers the manufacture of shapes, plates, and bars up to 1½ inches thick.

A8 steel is not classed as weldable. If agreed between the manufacturer and the purchaser, welding may be used to condition surface imperfections, using low-hydrogen type electrodes E10015 or E10016, together with not less than 212° F. preheat. Other specific instructions are contained in specification ASTM–A6. Specification A8 includes some tolerance tables which supersede similar provisions in ASTM–A6.

Structural silicon steel, ASTM–A94

Specification ASTM–A94 covers structural silicon steel used in special high-strength shapes, plates, and bars intended primarily for main stress-carrying members.

This steel is not classed as weldable. If agreed between the manufacturer and the purchaser, welding may be used to condition surface imperfections, using low-hydrogen

type electrodes E6015 or E6016, together with not less than 212° F. preheat. Other specific instructions are contained in specification A6.

A94 steel has been used since the 1920's in many bridges, large and small. For smaller structures, such as beam or girder spans, the A94 steel is used for main load-carrying members. For large structures, it is used for specific members, such as chords, web members, towers, and sometimes for floor systems, in order to reduce the dead-load weight. While A94 steel has served well in structures, it is not an ideal high-strength steel.

Its hardness causes difficulties in fabrication, resulting in many surface defects during rolling. A94 steel has a lower yield point than other high-strength steels, up to 1½-inch thickness. Its lump-sum fabrication cost is about 10 percent higher than A7 steel. It has lost its favor largely because of its high carbon content.

AREA high-strength steel

Committee 15 of the American Railway Engineering Association has recently presented a specification (*7*) covering high-strength shapes, plates, and bars intended primarily for use in main stress-carrying members. This specification was intended to supersede ASTM–A94. It is subject to the general requirements of specification ASTM–A6.

Cost data on this steel are not available for small or medium tonnages. It is understood this steel is not commercially available from all manufacturers.

Structural rivet steel, ASTM–A141

Specification ASTM–A141 covers soft carbon-steel rivets for structural purposes, and has been used since 1932. The material has proved to be very satisfactory.

High-strength structural rivet steel, ASTM–A195–52T

Specification ASTM–A195 covers high-strength rivet steel. Bars are manufactured into rivets either by the hot-heading process without annealing or by the cold-heading process with annealing at 1,450° F., and cooled slowly in the furnace or in still air.

Some difficulty has been experienced in the driving of these high-strength carbon-manganese rivets, in certain instances. For this reason, a new high-strength rivet steel specification has been published by the American Society for Testing Materials with the designation A406–57T, high-strength structural rivet steel.

Tests indicate that the new steel is free from the difficulties experienced with A195 steel, to which it is similar in physical properties, but radically different in chemical composition.

Quenched and tempered steel bolts and studs, ASTM–A325–55T

High-strength bolts have been in use since 1951. Their use is rapidly expanding, both for the replacement of rivets and, in some instances, for field connections in lieu of welding or riveting, where high strength is required.

High-strength bolts, studs, nuts, and washers are covered by ASTM–A325. Open-hearth or electric-furnace steel is used, except that acid Bessemer-process steel may be used for nuts.

Bolts up to 3 inches in diameter are covered by the specification. The physical properties required are obtained by heat treating under uniform conditions, quenching in a liquid medium, and tempering by reheating uniformly to not less than 800° F.

The nuts may not be heat treated but washers are either quenched and tempered or carburized, quenched, and tempered. For the bolts, the specified minimum content of both carbon and manganese is 0.30 percent each. For carburized washers, the specification requires carbon, 0.25 percent maximum, and manganese, 1.00 percent maximum.

A recent study by AREA Committee 15 (*8*) draws the following conclusions:

1. The bolted joints have proved superior to riveted joints.
2. There will be some loss in the clamping action of some bolts, possibly due to a reseating of the steel members.
3. Bolts tightened into the plastic range stay tight.
4. The bolts should be tightened to considerably higher values than the minimum recommended to provide for a subsequent loss in clamping action.
5. Extremely low temperatures have not adversely affected the bolts.
6. The enclosed area of the bolt shank and threads that have been properly installed will not rust.

High-strength, low-alloy structural steel, ASTM–A242

Specification ASTM–A242 covers high-strength, low-alloy structural steel shapes, plates, and bars for riveted and welded construction, intended primarily for use where savings in weight and resistance to atmospheric corrosion are important. To the specified limits of carbon and manganese, the manufacturer is required by the specification to add such alloying elements as necessary to give the specified mechanical properties. If the steel is purchased for welding, the suitability of the chemical composition for the welding process, under the given conditions, should be based upon evidence acceptable to the purchaser.

All of the high-strength steels have less ductility than A7 steel. This lower ductility is frequently recognized by the engineer specifying a higher grade of workmanship for fabrication. For example, all rivet holes in all thickness of material are subpunched and reamed or drilled full size in order to avoid incipient cracks around the edges of the holes, since such defects might act as stress-raisers and promote fatigue failures or brittle failures. Some engineers use more edge distance for rivets in high-strength steels. Flame-cut edges are considered satisfactory if flame softening by post-heating is used.

It should be noted that the high yield point for this and other high-strength steels, except that covered by ASTM specifications A8 and A94, varies with the thickness when over about ¾ inch. The high-strength steels, being the same modulus of elasticity as A7 steel, have less resistance to deflection and buckling because of the use of higher working stresses.

Thus the rules for stiffeners, slenderness ratios, and ratios of plate width to thickness are more restricted. Compression flanges for composite construction buckle more easily and need special attention, particularly for erection. Rivets should preferably be not less than 1 inch in diameter. In cases where total thickness of parts riveted is ¾ inch or less, smaller size rivets may be desirable. Tack welds should be made with low-hydrogen electrodes. The A242 steels are more difficult to draw up tight before riveting or welding than the A7 steel. Fabrication costs are said to be about 5 percent greater.

The high-strength carbon steels cost 20 to 30 percent more per pound than A7 steel. However, it will often be found that these high-strength steels are more economical for movable bridges, the suspended spans of cantilever bridges, and other large and heavy structures, because of the lesser amount of steel required and the resultant savings in material cost and freight charges.

Military Specifications

Mention should be made of military specifications for high-strength steel, which are somewhat similar to the high-strength steels covered by the ASTM–A94 and A242 specifications. These are MIL–S–20166 for shapes and bars (*9*) and MIL–S–16113 for plates (*10*).

These military steels are of excellent weldability. Their cost is high because of the special manufacturing process involved.

Proprietary Steels

The proprietary steels discussed here are not covered by ASTM standard specifications, and hence must be ordered from the manufacturers by individual specification or on the basis of the manufacturer's specification.

The requirements should be included that the steel shall be manufactured by the open-hearth or electric-furnace processes. The general requirements of the ASTM–A6 specification apply.

Medium Manganese and Man-Ten A242 steels

Medium Manganese steel, manufactured by the Bethlehem Steel Company, and Man-Ten A242 steel, manufactured by the United States Steel Corporation, are high-strength steels that have been widely used for riveted structures, being in the strength class of A94 silicon steel. They meet the physical requirements, but not the chemical requirements, of the ASTM–A242 steel specification. Their cost is considerably less than that of ASTM–A242 steel, and even less than that of A94 steel.

These steels are not classed as weldable steels, although secondary welding, such as for surface conditioning, is more acceptable than for A94 steel. The welding of shear developers

may be considered permissible for compression flanges. These steels contain copper and are very satisfactory for high-strength riveted steel bridge construction where corrosion resistance is important. Although the specifications of the two companies are not exactly alike, they are nearly so.

Manganese Vanadium and Tri-Ten steels

Manganese Vanadium steel, manufactured by the Bethlehem Steel Company, and Tri-Ten steel, manufactured by the United States Steel Corporation, both comply with the requirements for ASTM–A242 steels and cost about the same as other A242 steels. They also practically meet the specification MIL–S–12505 (CE) (*11*).

These steels are recommended for use where a high-strength steel of weldable quality is required. They contain copper and are satisfactory for welded bridge construction where high strength and corrosion resistance are desirable.

Mayari R and Cor-Ten steels

Mayari R steel, manufactured by the Bethlehem Steel Company, is said to be satisfactory for welding, providing the customer stipulates that the steel is to be welded. Cor-Ten steel, manufactured by the United States Steel Corporation, is said by the manufacturer to have satisfactory weldability for thicknesses up to ½ inch and will have tensile strengths equal to or in excess of the base metal; metal over ½ inch thick is to be used for riveted construction.

These steels are recommended for use where a high-strength steel and resistance to corrosion are desired. Both steels contain copper, nickel, and chromium and are therefore more resistant to corrosion than the high-strength steels discussed previously.

Quenched and tempered high-strength steel, T1

The very-high-strength steel designated as T1 by its manufacturers is not recommended for use unless properly heat treated. In this condition, it possesses toughness and weldability considerably above that usually expected at this high-strength level. It has excellent welding properties; low-hydrogen type electrodes are used, the strength or class depending upon the subsequent heat treatment.

This steel also has a high resistance to atmospheric corrosion, about four times that of carbon steel. It may have a maximum hardness of 410 VPN [2] in the heat-affected zone and may require special equipment for some fabrication operations. Preheating before flame cutting is also necessary.

The cost of T1 steel is high, yet it has been used to advantage for the very highly stressed members in large bridges or other structures. Its use at present is largely for welded pressure vessels and earth-moving equipment. T1 steel is furnished in the form of plates, bars, forgings, and semifinished products.

[2] Vickers diamond pyramid hardness number as specified in Federal Test Method Standard No. 151, Metals.

References

(1) 1957 Supplement to Book of ASTM Standards, Part I, Ferrous Metals.

(2) 1956 Supplement to Book of ASTM Standards, Part I, Ferrous Metals.

(3) 1955 Book of ASTM Standards, Part I, Ferrous Metals.

(4) Steel Products Manual, Alloy Steel, American Iron and Steel Institute, July 1955, p. 5.

(5) Steel Products Manual, High Strength Low Alloy Steel, American Iron and Steel Institute, May 1955, p. 5.

(6) Standard Specifications for Welded Highway and Railway Bridges, 1956, American Welding Society.

(7) Proceedings of the American Railway Engineering Association, vol. 58, p. 687.

(8) *Use of High-Strength Structural Bolts in Steel Railway Bridges.* Report of Committee 15—Iron and Steel Structures. American Railway Engineering Association Bulletin vol. 56, No. 520, Jan. 1955, p. 598.

(9) Military Specification (Navy), MIL-S–20166, Oct. 3, 1951. Steel: Bars and shapes (for hull construction) (including material for drop and miscellaneous forgings).

(10) Military Specification (Navy), MIL-S–16113B, Dec. 4, 1953. Steel plate, hull and ordnance, structural, black (uncoated), and zinc-coated (galvanized).

(11) Military Specification MIL–S–12503 (CE), Mar. 2, 1953. Steel: Structural, low-alloy, for welded engineer equipment.

Motor-Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in a table on pages 90–91. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of July 1, 1958, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit trucks, truck-tractor semitrailer combinations, and other combinations.

U. S. GOVERNMENT PRINTING OFFICE: 1958

UNITED STATES
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VOL. 30, NO. 5

DECEMBER 1958

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 5 *December 1958*

C. M. Billingsley, *Editor*

BUREAU OF PUBLIC ROADS

Washington 25, D. C.

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IN THIS ISSUE

Travel Patterns in 50 Cities 105

Observations Concerning Urban Traffic Volume Patterns in Tennessee 123

Traffic Article Postponed 131

PUBLIC ROADS is sold by the Superintendent of Documents, Government Printing Office, Washington 25, D. C., at $1 per year (25 cents additional for foreign mailing) or 20 cents per single copy. Free distribution is limited to public officials actually engaged in planning or constructing highways, and to instructors of highway engineering. There are no vacancies in the free list at present.

Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

U. S. DEPARTMENT OF COMMERCE
LEWIS L. STRAUSS, Secretary

BUREAU OF PUBLIC ROADS
BERTRAM D. TALLAMY, Administrator
ELLIS L. ARMSTRONG, Commissioner

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Travel Patterns in 50 Cities

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by FRANK B. CURRAN, Statistician, and JOSEPH T. STEGMAIER, Highway Research Engineer

During the past 15 years origin-and-destination traffic surveys of the home-interview type have been conducted in more than one hundred cities. This article presents information regarding the purpose for which trips were made by residents in 50 of these urban areas and the mode of travel they used. Data are also included pertaining to basic household characteristics of the areas such as the numbers of dwelling units, residents, and automobiles owned, and the relations between these characteristics and the volume of trips classified according to purpose and mode of travel. The urban areas have been grouped by population size to disclose whatever travel trends or patterns may exist among cities in the several population groups.

The percentage distribution of major trip purposes is fairly uniform in cities of all sizes. Analysis by mode of travel, however, shows a variable pattern. The proportion of trips by automobiles and taxis increases as city size decreases. On the whole, mass transit is by far the most prevalent mode of travel in the largest cities, but its relative importance varies depending upon the trip purpose. Trips for social and recreational purposes, for instance, generally involve the use of automobiles.

In most cases, the volume of daily trips by residents within an urban area is directly related to the numbers of persons, dwelling units, and automobiles registered in the area. The relations vary, however, depending upon the trip purpose and mode of travel.

AMONG the more important factors affecting the planning of streets and highways are the means by which residents travel within the city, the purposes for which the trips are made, and the relations between these trips and residential characteristics such as the number of persons living in the area, the number of dwelling units they occupy, and the number of automobiles they own. At the time this article was prepared, information of this sort was available from origin-and-destination traffic studies[1] of the home-interview type which had been made in 101 urban areas since 1944. The product of these studies includes a great mass of data on the local travel habits of urban residents on an average weekday during the period of the survey.

Data from these studies have already been analyzed and the results have been put to use in each of the individual urban areas surveyed. However, knowledge of the general or average pattern for groups of cities of similar size should be very beneficial to highway planners. Thus it may be possible to establish norms that might be helpful in anticipating the changes which will take place in the traffic patterns of a city as the pattern of living changes.

The primary intent of this article, therefore, is to call attention to the more significant aspects of the many-sided travel patterns for 50 of these cities, considered either singly or in combination. Information from the recent past regarding travel habits of city residents should be valuable to urban planners, highway engineers, and economists in attacking the transportation problems of the present and future. It is also hoped that the article will serve to call attention to the quantity and quality of data that have become available as a result of such surveys. A list of the selected cities showing survey dates and population at the time of the study is given in table 1. It should be noted that almost one-third of the studies were conducted during the latter part of World War II and the year following the end of the war. Some of the variations in trip-purpose and travel-mode patterns which are discussed later may be associated with the year of the basic survey or the geographical location of the study area.

Table 1.—Population and period of survey in 50 urban areas

Urban area	Population	Period of survey
Albuquerque, N. Mex.	116,056	June 1949–July 1949.
Altoona, Pa.	85,347	July 1950–Sept. 1950.
Appleton, Wis.	39,172	June 1953–July 1953.
Baltimore, Md.	912,809	Sept. 1945–Oct. 1945.
Bay City, Mich.	69,231	July 1948–Oct. 1948.
Charleston, S. C.	73,205	Feb. 1947–Mar. 1947.
Chester, Pa.	127,408	June 1951–Oct. 1951.
Columbus, Ga.	79,192	Oct. 1945–Dec. 1945.
Dallas, Tex.	533,606	Nov. 1950–Mar. 1951.
Duluth, Minn.-Superior, Wis.	130,847	May 1948–June 1948.
Fargo, N. Dak.-Moorhead, Minn.	49,852	June 1949–Aug. 1949.
Grand Rapids, Mich.	220,977	July 1947–Oct. 1947.
Harrisburg, Pa.	103,303	June 1946–Sept. 1946.
Honolulu, T. H.	214,236	Apr. 1947–Sept. 1947.
Houston, Tex.	878,629	Mar. 1953–June 1953.
Johnstown, Pa.	87,709	July 1949–Sept. 1949.
Kalamazoo, Mich.	72,024	Apr. 1946–May 1946.
Lansing, Mich.	122,776	Sept. 1946–Nov. 1946.
Macon, Ga.	77,665	July 1946–Aug. 1946.
Madison, Wis.	104,074	May 1949–June 1949.
Muskegon, Mich.	83,721	July 1946–Aug. 1946.
Newark, N. J.	1,456,947	Aug. 1945–Jan. 1946.
Norfolk, Va.	335,910	June 1950–Aug. 1950.
Norristown, Pa.	39,185	June 1949–Aug. 1949.
Philadelphia, Pa.	2,233,531	June 1947–Nov. 1947.
Phoenix, Ariz.	161,567	Nov. 1946–Feb. 1947.
Pontiac, Mich.	79,431	Apr. 1947–May 1947.
Portland, Oreg.	153,128	July 1946–Sept. 1946.
Racine, Wis.	78,033	Aug. 1949–Oct. 1949.
Reading, Pa.	119,851	Nov. 1946–Dec. 1946.
Rockford, Ill.	116,000	July 1950–Aug. 1950.
Sacramento, Calif.	201,345	Dec. 1947–May 1948.
Saginaw, Mich.	112,902	July 1948–Sept. 1948.
St. Louis, Mo.	974,545	Apr. 1945–July 1945.
St. Paul-Minneapolis, Minn.	915,960	May 1949–Nov. 1949.
Salt Lake City, Utah	195,571	June 1946–Sept. 1946.
San Francisco, Calif.	1,468,033	July 1946–Dec. 1946.
San Juan, P. R.	312,069	June 1948–July 1948.
Scranton, Pa.	137,089	June 1950–Aug. 1950.
Seattle, Wash.	518,563	May 1946–Aug. 1946.
Sharon-Farrell, Pa.	48,432	June 1949–July 1949.
Spokane, Wash.	138,381	July 1946–Dec. 1946.
Tacoma, Wash.	138,700	June 1948–Aug. 1948.
Tucson, Ariz.	126,900	Mar. 1948–Apr. 1948.
Washington, D. C.	1,109,860	May 1948–Sept. 1948.
Wichita, Kans.	238,302	Nov. 1951–Apr. 1952.
Williamsport, Pa.	55,216	July 1954–Aug. 1954.
Wilmington, Del.	181,445	Apr. 1948–July 1948.
Wisconsin Rapids, Wis.	16,504	Sept. 1950–Oct. 1950.
York, Pa.	77,350	July 1951–Aug. 1951.

[1] *Traffic planning studies in American cities*, by John T. Lynch. PUBLIC ROADS, vol. 24, No. 6, Oct.–Nov.–Dec. 1945. The procedures used in these studies are given in greater detail in the *Manual of Procedures for Home Interview Traffic Study*, which is available by purchase from the Public Administration Service, 1313 East 60th Street, Chicago, Ill.

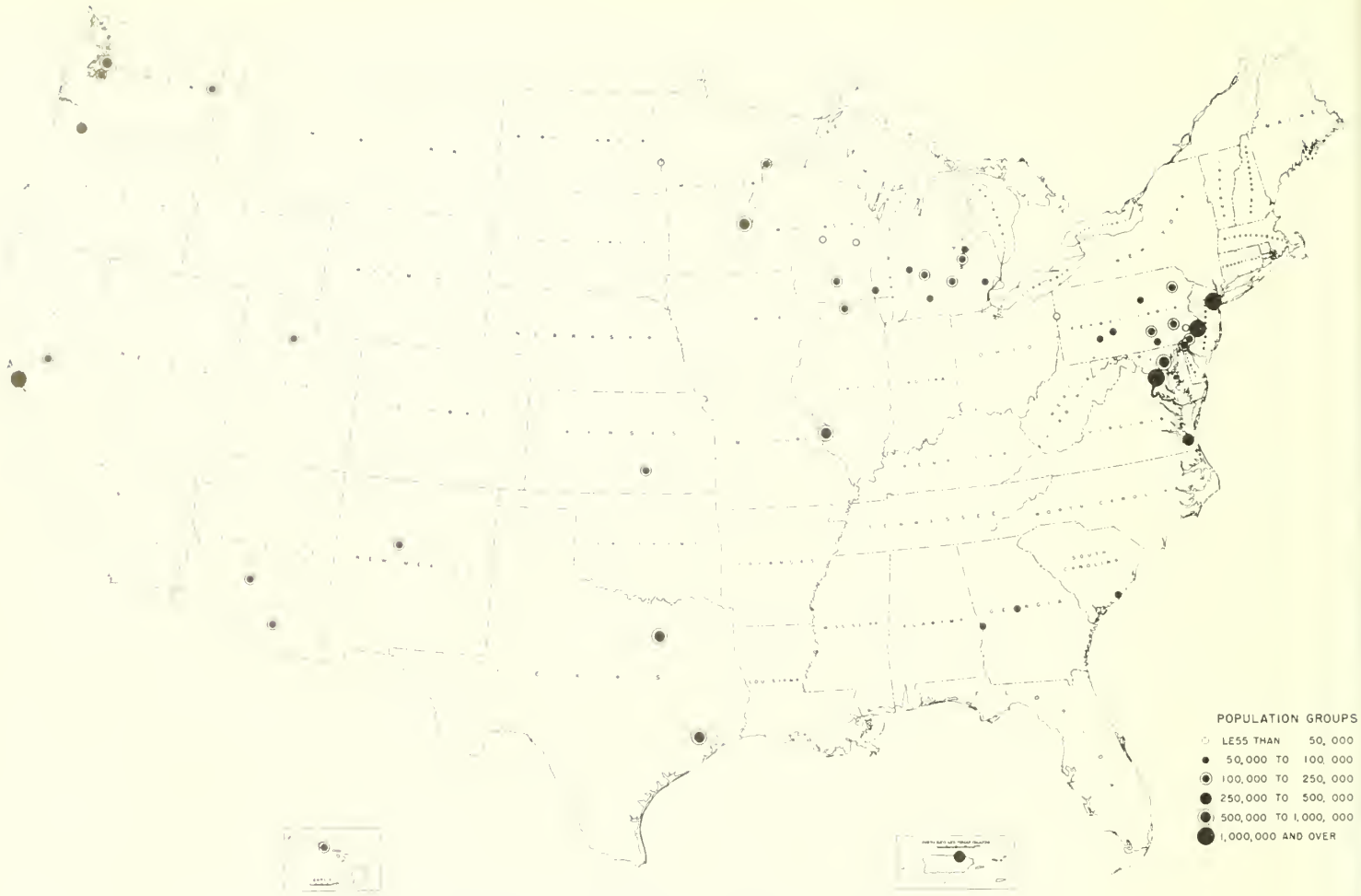

Figure 1.—Geographical distribution of the 50 cities included in study.

Definitions

The *urban areas* referred to in this article are the areas within which the basic surveys were conducted. They generally include the central city as well as any portion of the contiguous built-up area that may exist beyond the corporate limits. Their boundaries are usually delimited by an imaginary line called the external cordon. These areas resemble but do not coincide with *urbanized areas* as defined by the Bureau of the Census. In this article the terms *urban area* and *city* are used interchangeably.

A *trip* is defined as a one-way movement in a vehicle by a resident of the urban area. There are no round trips but rather two or more one-way trips. The only trips considered here are *internal trips*, so called because both origin and destination are within the boundaries of the survey area. *External trips* to or from points beyond the external cordon are not included. The external phase of the basic surveys was concerned only with automobile travel beyond the cordon and only automobile-driver trip information was included. These external automobile-driver trips amounted to about 5 percent of the total internal and external automobile-driver trips in the largest urban areas and about 45 percent in the smallest cities included in this study.

As the term is used in these surveys, *mode of travel* depends upon (1) the type of vehicle used (automobile, taxi, truck, or mass-transit vehicle), and (2) the status of the user (driver or passenger). The modes of travel recorded in most of the individual surveys were as follows: automobile drivers, automobile passengers, taxi passengers, truck passengers, bus or streetcar passengers, railroad passengers, and passengers in other mass-transit vehicles. For purposes of analysis, some of these modes have been combined.

The term *purpose of trip* is used in its obvious sense to explain why a person made the trip. However, for every internal trip recorded, the survey data show not only why the traveler went to his destination (purpose to), but also why he had been at the point of origin (purpose from). The purposes (both to and from) were originally ten: work, business, medical-dental, school, social-recreational, eat meal, shop, change mode of travel, serve passenger, and home. However, as with modes of travel, some of the trip purposes have been combined.

Household characteristics include the numbers of persons, dwelling units, automobiles owned, and persons 5 years of age and over. *Dwelling unit* is used in the sense of the Bureau of the Census—"In general, . . . a group of rooms or a single room occupied or intended for occupancy as separate living quarters by a family or other group of persons living together or by a person living alone."

Scope of Article

Although at the time of this analysis over one hundred comprehensive urban traffic surveys had been completed, trip purpose-to-

Table 2.—Distribution by population groups of all urbanized areas, of urban areas where origin and destination studies have been made, and of urban areas included in the present study

Urban area population groups	All urbanized areas, 1950 census		Urban areas with completed O & D studies		Urban areas included in this study	
	Number	Percent	Number	Percent	Number	Percent
Over 1,000,000	12	7.6	6	5.9	4	8.0
500,000–1,000,000	13	8.3	11	10.9	6	12.0
250,000–500,000	24	15.3	9	8.9	3	6.0
100,000–250,000	70	44.6	43	42.6	20	40.0
50,000–100,000	38	24.2	22	21.8	12	24.0
Less than 50,000	------	------	10	9.9	5	10.0
Total	157	100.0	101	100.0	50	100.0

purpose tabulations had been prepared in only 50 cities with sufficient uniformity to permit summarizing the results by city groups. These 50 cities seem to provide a sufficiently good distribution among the population groups studied so that the data are representative.

Figure 1 shows the geographical distribution of the selected cities by population groups. The 50 cities accounted for 10.8 percent of the

Table 3.—Number of trips by each mode of travel in Madison, Wis., classified according to trip purpose

Trips from—	Trips to—										
	Work	Business	Medical-dental	School	Social-recreation	Eat meal	Shop	Change mode of travel	Serve passengers	Home	Total
Automobile Drivers											
Work	8,008	663	89	41	522	3,717	990		1,242	11,214	26,486
Business	623	828	10	30	301	190	468		210	2,164	4,824
Medical-dental	49	20	10		31	9	119		50	271	559
School	72	20	10	119	139	290	40		150	1,092	1,932
Social-recreation	142	190	40	20	1,377	258	401	10	1,206	6,395	10,039
Eat meal	3,167	171		169	290		101		439	791	5,128
Shop	270	251	29	31	598	139	1,365	11	321	5,333	8,348
Change mode of travel		10	10		10	20	21		20	148	239
Serve passengers	1,924	300	70	209	926	349	620	20	1,776	5,827	12,021
Home	12,648	2,624	341	1,331	5,749	691	4,234	99	6,953		34,670
Total	26,903	5,077	609	1,950	9,943	5,663	8,359	140	12,367	33,235	104,246
Automobile Passengers											
Work	191	63	30		203	503	196	71		3,971	5,228
Business	41	217	30	10	184	20	153			725	1,380
Medical-dental		21		10	52		72			303	458
School	20	52	53	82	256	188	20			871	1,542
Social-recreation	92	82	10	89	3,739	297	451	101		10,048	14,909
Eat meal	441	10		267	251		20			690	1,679
Shop	29	63	10	11	535	62	708	10		2,673	4,101
Change mode of travel	30				20	10	22	10		271	363
Serve passengers											
Home	4,533	901	313	2,676	9,384	558	2,421	285			21,071
Total	5,377	1,409	446	3,145	14,624	1,638	4,063	477		19,552	50,731
Streetcar and Bus Passengers											
Work	130	69	20	10	140	504	220	175		7,429	8,697
Business	30	60		20	30		50	10		805	1,005
Medical-dental		10		10	20			20		300	360
School	161	40	20	90	180	553	201	60		5,213	6,518
Social-recreation	20	41	20	20	159	40	70	41		3,258	3,669
Eat meal	434	20		454	40		20			363	1,331
Shop	40	49	10	50	131		150	40		2,522	2,992
Change mode of travel	110	10		59	70	10	59	40		161	519
Serve passengers											
Home	8,153	1,118	382	5,556	3,238	361	1,730	210			20,748
Total	9,078	1,417	452	6,269	4,008	1,468	2,500	596		20,051	45,839
Taxi Passengers											
Work	41	20	50		30	10	10			413	574
Business	10	10		10	20	10				149	209
Medical-dental	10				10					131	151
School				10	11	20				121	162
Social-recreation	21	11			79	20				481	612
Eat meal	20				10		10			40	80
Shop	30						40			90	160
Change mode of travel	10				20					110	140
Serve passengers											
Home	804	139	110	239	500	60	102	89			2,043
Total	946	180	160	259	680	120	162	89		1,535	4,131
Truck Passengers											
Work	92									31	123
Business											
Medical-dental											
School											
Social-recreation										10	10
Eat meal											
Shop											
Change mode of travel											
Serve passengers											
Home	11				10						21
Total	103				10					41	154
All Modes of Travel											
Work	8,462	815	189	51	895	4,734	1,416	246	1,242	23,058	41,108
Business	704	1,115	40	70	535	220	671	10	210	3,843	7,418
Medical-dental	59	51	10	20	113	9	191	20	50	1,005	1,528
School	253	112	83	301	586	1,051	261	60	150	7,297	10,154
Social-recreation	275	324	70	129	5,354	615	922	152	1,206	20,192	29,239
Eat meal	4,062	201		890	591		151		439	1,884	8,218
Shop	369	363	49	92	1,264	201	2,263	61	321	10,618	15,601
Change mode of travel	150	20	10	59	120	40	102	50	20	690	1,261
Serve passengers	1,924	300	70	209	926	349	620	20	1,776	5,827	12,021
Home	26,149	4,782	1,146	9,802	18,881	1,670	8,487	683	6,953		78,553
Total	42,407	8,083	1,667	11,623	29,265	8,889	15,084	1,302	12,367	74,414	205,101

Table 1.—Number and percentage of trips by each mode of travel in 50 cities, classified according to trip purpose

Mode of travel	Trip purpose											
	Work and business		Social and recreation		Shop		Miscellaneous		Home		Total	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Automobile driver	3,679,848	13.2	1,079,942	3.9	910,831	3.3	1,524,373	5.5	4,187,918	15.1	11,382,912	41.0
Automobile and taxi passengers	1,065,361	3.9	1,520,382	5.5	488,798	1.6	486,546	1.7	2,634,629	9.5	6,195,716	22.2
Mass transit passenger	3,014,103	10.8	736,487	2.6	690,435	2.6	1,270,461	4.6	4,487,541	16.2	10,199,027	36.8
Total	7,759,312	27.9	3,336,811	12.0	2,090,064	7.5	3,281,380	11.8	11,310,088	40.8	27,777,655	100.0

total United States population in 1950, and 16.8 percent of the urban population. As table 2 indicates, the distribution of the 50 cities by population groups among the 157 urbanized areas of the 1950 census is only fair, but it follows very closely the group distribution of the 101 cities from which origin-destination traffic survey data were available.

The present analyses have been limited to two questions: how and why residents make their trips within an urban area. It does not consider two other important questions which relate to the origin and destination of trips within the area. Although these data are available for each city, records of trips from place to place within a city cannot justifiably be combined for more than one city at a time, because it is difficult to relate areas when so little is known about their land-use characteristics.

The process of summarizing data to discover travel patterns, related to purpose of trip and mode of travel, began with the cities where the surveys were made. In each of the 50 cities the procedures recommended in the *Manual of Procedures for Home Interview Traffic Study* were generally followed, and tables were compiled in which trips were classified uniformly by mode of travel and purpose of trip. One tabulation was prepared for each mode of travel, showing the number of trips from each purpose to each purpose. However, the number of travel modes reported in different cities varied; trips by train passengers were reported only in 2 cities and trips by "other" passengers were reported only in 5 cities. A typical example of the basic tabulations for an individual city is presented in table 3.

In the course of assembling and combining the data from different cities it became evident that certain less significant trip purposes and travel modes might advantageously be combined. On the average, the 5 least important trip purposes accounted for less than 12 percent of the total number of trips, and not one of these purposes accounted for as much as 4 percent. These categories were combined to form a miscellaneous group.

Minor trip purposes and the percentages of trips accounted for by each were as follows: to serve passenger, 3.4 percent, change mode of travel, 3.3 percent, school, 2.3 percent, eat meal, 1.7 percent, and medical-dental, 1.1 percent.

In addition, since business trips amounted to less than 5 percent of all internal trips and were often difficult to dissociate from work trips, the two were combined as work and business trips. Thus, the five major trip purposes were work and business, social-recreational, shopping, miscellaneous, and home.

An examination of the data for the 7 modes of travel indicated that 4 modes accounted for less than 2 percent of all trips, and not 1 of the 4 modes accounted for as much as 1 percent of the trips. The least important travel modes were as follows: taxi passengers, 0.8 percent, train passengers, 0.7 percent, truck passengers, 0.2 percent, and other passengers, 0.2 percent.

Since these travel modes represented such small proportions of the total, they were combined with other modes of similar characteristics. Taxi and truck passengers were combined with automobile passengers, whereas passengers using trains or other interurban facilities such as subways, ferries, or highway buses were combined with streetcar and bus passengers. Thus, three major modes of travel appeared to be sufficiently representative: automobile drivers, automobile and taxi passengers, and mass-transit passengers.

Although the trip purposes and travel modes that have lost their identity through summarization are relatively insignificant in the total travel pattern, they may be important

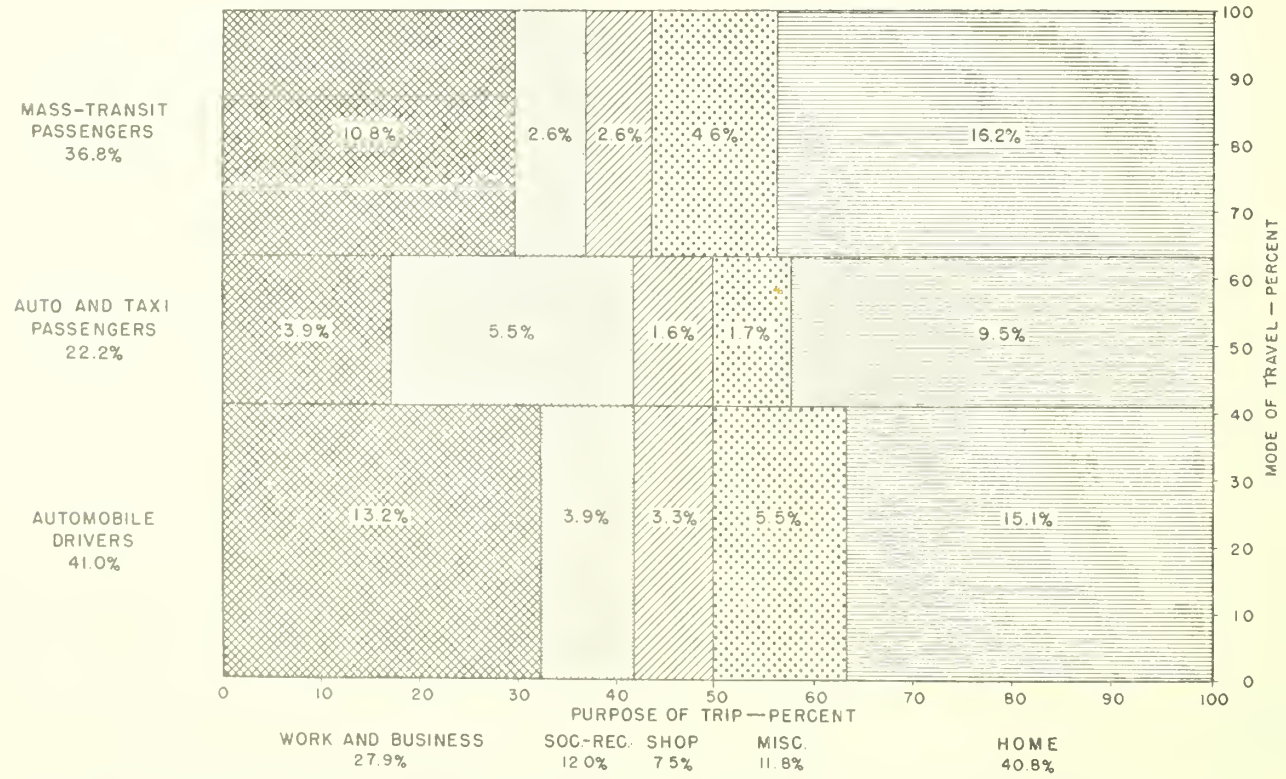

Figure 2.—Percentage distribution of trips according to purpose, and further classified by mode of travel.

Table 5.—Number and percentage of trips by each mode of travel in six population groups, classified according to trip purpose

Mode of travel	Population group	Number of cities	Trip purpose: Work and business		Social and recreation		Shop		Miscellaneous		Home		Total	
			Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
Automobile drivers	1,000,000 and over	4	1,143,303	35.4	300,892	9.3	201,334	6.2	414,396	12.9	1,165,651	36.2	3,225,576	100.0
Automobile and taxi passengers			344,013	19.1	420,539	23.3	116,687	6.5	151,320	8.4	769,472	42.7	1,802,031	100.0
Mass-transit passengers			1,401,980	28.2	345,528	7.0	309,684	6.2	884,034	17.8	2,027,099	40.8	4,968,325	100.0
Total			2,889,296	28.9	1,066,959	10.7	627,705	6.3	1,449,750	14.5	3,962,222	39.6	9,995,932	100.0
Automobile drivers	500,000–1,000,000	6	1,110,178	31.5	309,475	8.8	296,569	8.4	438,527	12.4	1,370,171	38.9	3,524,920	100.0
Automobile and taxi passengers			289,710	16.6	402,040	23.0	145,159	8.3	156,448	9.0	753,517	43.1	1,746,874	100.0
Mass-transit passengers			983,146	33.0	184,998	6.2	201,002	6.8	190,078	6.4	1,418,248	47.6	2,977,472	100.0
Total			2,383,034	28.9	896,513	10.9	642,730	7.8	785,053	9.5	3,541,936	42.9	8,249,266	100.0
Automobile drivers	250,000–500,000	3	234,358	33.9	72,134	10.4	58,241	8.4	84,678	12.2	242,866	35.1	692,277	100.0
Automobile and taxi passengers			70,884	16.9	110,151	26.3	34,775	8.3	31,456	7.5	172,196	41.0	419,462	100.0
Mass-transit passengers			175,776	30.2	52,309	9.0	42,057	7.2	60,348	10.4	251,271	43.2	581,761	100.0
Total			481,018	28.4	234,594	13.8	135,073	8.0	176,482	10.1	666,333	39.4	1,693,500	100.0
Automobile drivers	100,000–250,000	20	888,964	30.4	287,031	9.8	270,770	9.3	433,692	14.8	1,042,586	35.7	2,923,043	100.0
Automobile and taxi passengers			264,644	16.0	428,819	25.9	146,796	8.9	116,747	7.1	696,795	42.1	1,653,801	100.0
Mass-transit passengers			330,220	26.3	115,567	9.2	104,161	8.3	108,051	8.6	596,264	47.6	1,254,263	100.0
Total			1,483,828	25.4	831,417	14.3	521,727	8.9	658,490	11.3	2,335,645	40.1	5,831,107	100.0
Automobile drivers	50,000–100,000	12	242,565	30.0	87,961	10.9	65,073	8.0	117,711	14.6	294,964	36.5	808,274	100.0
Automobile and taxi passengers			79,696	17.5	120,083	26.3	35,941	7.9	24,855	5.5	195,281	42.8	455,856	100.0
Mass-transit passengers			110,197	29.8	33,696	9.1	28,778	7.8	24,280	6.6	172,236	46.7	369,187	100.0
Total			432,458	26.5	241,740	14.8	129,792	7.9	166,846	10.2	662,481	40.6	1,633,317	100.0
Automobile drivers	Less than 50,000	5	60,480	29.0	22,449	10.8	18,844	9.0	35,369	16.9	71,680	34.3	208,822	100.0
Automobile and taxi passengers			16,414	13.9	38,750	32.9	9,440	8.0	5,720	4.9	47,368	40.3	117,692	100.0
Mass-transit passengers			12,784	26.6	4,389	9.2	4,753	9.9	3,670	7.6	22,423	46.7	48,019	100.0
Total			89,678	24.0	65,588	17.5	33,037	8.8	44,759	11.9	141,471	37.8	374,533	100.0
Automobile drivers	All groups	50	3,679,848	32.3	1,079,942	9.5	910,831	8.0	1,524,373	13.4	4,187,918	36.8	11,382,912	100.0
Automobile and taxi passengers			1,065,361	17.2	1,520,382	24.5	488,798	7.9	486,546	7.9	2,634,629	42.5	6,195,716	100.0
Mass-transit passengers			3,014,103	29.5	736,487	7.2	690,435	6.8	1,270,461	12.5	4,487,541	44.0	10,199,027	100.0
Total			7,759,312	27.9	3,336,811	12.0	2,090,064	7.5	3,281,380	11.8	11,310,088	40.8	27,777,655	100.0

under certain conditions and in individual cities. Further comments are given in the appendix on page 120.

Summary for 50 Cities

All of the internal trips by residents of the 50 urban areas have been combined in table 4 and classified according to the 5 purposes and 3 modes of travel. Of the total trips numbering almost 28 million, trips by automobile drivers accounted for the largest share and were followed in order by mass-transit passengers and automobile and taxi passengers. Homeward-bound trips predominated among the five major trip purposes; work and business trips ranked second, and were followed by social-recreational, miscellaneous, and shopping trips in that order.

Although automobile drivers represented the predominant travel mode, mass-transit passengers traveling home constituted the largest mode-purpose category, accounting for nearly one-sixth of all trips. Homeward-bound automobile drivers followed closely with 15 percent of all trips; automobile drivers on work and business trips, 13 percent; mass-transit passenger trips to work and business, 11 percent; and automobile and taxi passengers on their way home, 9 percent. The remaining individual mode-purpose categories accounted for 5 percent or less of the total trips.

These percentage distributions of the total trips by purpose and mode of travel are also shown in figure 2. In this chart the area of each rectangle and the percentage shown

Table 6.—Percentage of trips for each trip purpose in six population groups, classified according to mode of travel

Purpose of trip	Population group	Number of cities	Mode of travel: Automobile drivers	Automobile and taxi passengers	Mass-transit passengers	Total
Work and business	1,000,000 and over	4	39.6	11.9	48.5	100.0
Social and recreation			28.2	39.4	32.4	100.0
Shop			32.1	18.6	49.3	100.0
Miscellaneous			28.6	10.4	61.0	100.0
Home			29.4	19.4	51.2	100.0
All purposes			32.3	18.0	49.7	100.0
Work and business	500,000–1,000,000	6	46.6	12.2	41.2	100.0
Social and recreation			34.5	44.9	20.6	100.0
Shop			46.1	22.6	31.3	100.0
Miscellaneous			55.9	19.9	24.2	100.0
Home			38.7	21.3	40.0	100.0
All purposes			42.7	21.2	36.1	100.0
Work and business	250,000–500,000	3	48.7	14.7	36.6	100.0
Social and recreation			30.7	47.0	22.3	100.0
Shop			43.1	25.7	31.2	100.0
Miscellaneous			48.0	17.8	34.2	100.0
Home			36.5	25.8	37.7	100.0
All purposes			40.9	24.8	34.3	100.0
Work and business	100,000–250,000	20	59.9	17.8	22.3	100.0
Social and recreation			34.5	51.6	13.9	100.0
Shop			51.9	28.1	20.0	100.0
Miscellaneous			65.9	17.7	16.4	100.0
Home			44.7	29.8	25.5	100.0
All purposes			50.1	28.4	21.5	100.0
Work and business	50,000–100,000	12	56.1	18.4	25.5	100.0
Social and recreation			36.4	49.7	13.9	100.0
Shop			50.1	27.7	22.2	100.0
Miscellaneous			70.6	14.9	14.5	100.0
Home			44.5	29.5	26.0	100.0
All purposes			49.5	27.9	22.6	100.0
Work and business	Less than 50,000	5	67.4	18.3	14.3	100.0
Social and recreation			34.2	59.1	6.7	100.0
Shop			57.0	28.6	14.4	100.0
Miscellaneous			79.0	12.8	8.2	100.0
Home			50.7	33.5	15.8	100.0
All purposes			55.8	31.4	12.8	100.0
Work and business	All groups	50	47.4	13.7	38.9	100.0
Social and recreation			32.4	45.5	22.1	100.0
Shop			43.6	23.4	33.0	100.0
Miscellaneous			46.5	14.8	38.7	100.0
Home			37.0	23.3	39.7	100.0
All purposes			41.0	22.2	36.8	100.0

therein represent the relation of the number of trips in each mode-purpose category to the total number of trips. Upon examining the horizontal bars for each mode of travel, it is seen at a glance that a larger proportion of automobile drivers were on work and business trips than mass-transit passengers, but relatively more transit passengers were going home. Also, proportionately many more automobile and taxi passengers were on social and recreational trips than was the case with either of the other two modes of travel. This chart, of course, is not typical of any particular city, but represents all of the trips made in the 50 urban areas by residents of these areas on a typical weekday during the various periods studied.

Distribution of Trips by Population Groups

In table 5 the trips are shown by population groups of the urban areas in which they were made. Almost 10 million trips are accounted for in the 1 million and over population group and nearly 375,000 trips in the smallest group, which is for cities of less than 50,000 population. The number of trips in the various purpose-mode cells ranged from less than 4,000 to more than 2 million.

Purpose distribution

A pattern of uniformity for trip purposes among all population groups is observed in table 5. Generally, there was no pronounced trend in the purpose distribution of trips from one population group to another. Exceptions to this observation were an increase in the proportion of social and recreational trips and a slight reduction in the percentage of work and business trips, as

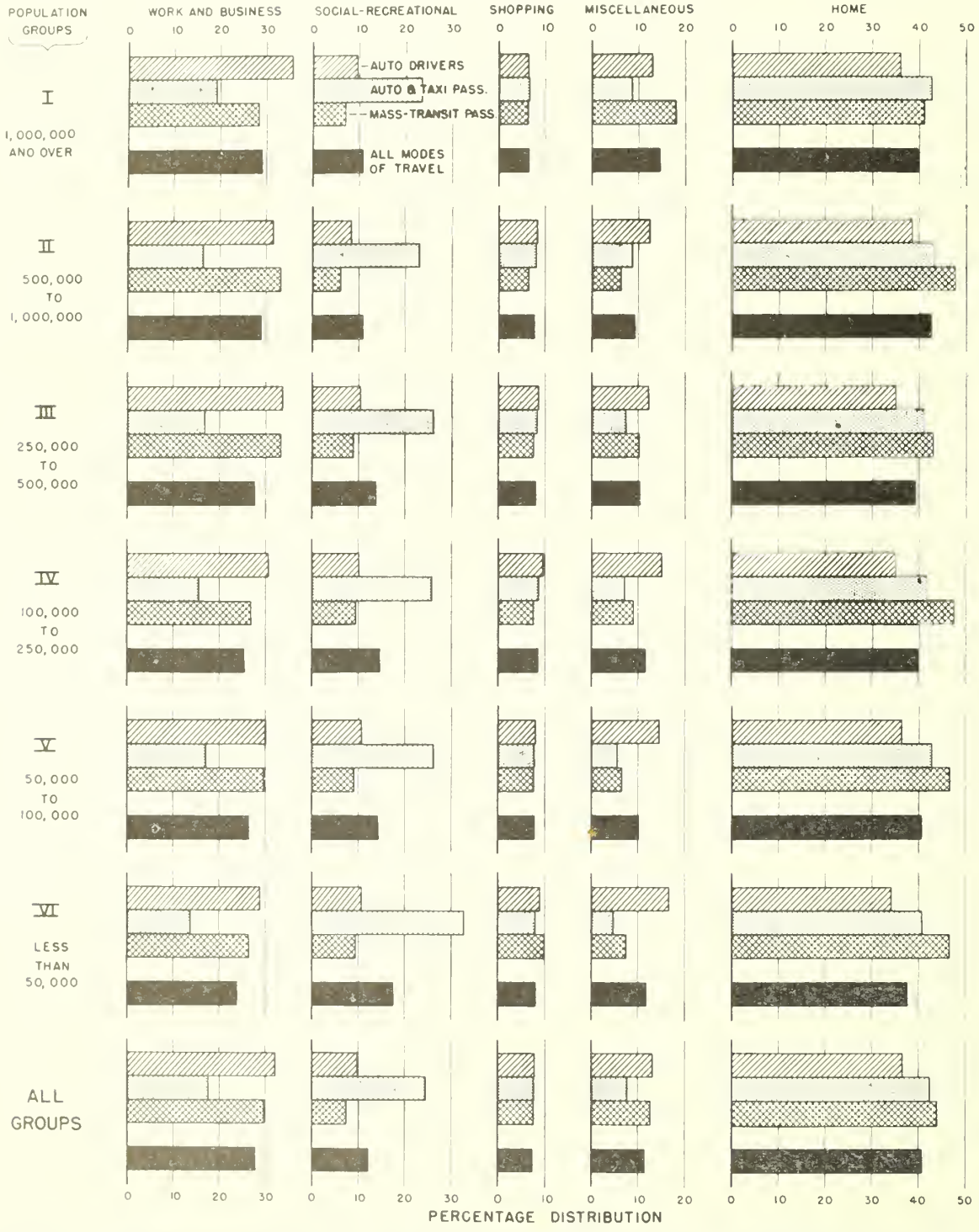

Figure 3.—Percentage distribution of trips according to purpose, and further classified by mode of travel and population group.

population decreases, but these trends did not hold for the individual modes of travel.

The slight effect that city size apparently has on the percentage distribution of trips by purpose is portrayed in figure 3. For each of the six population groups and for each mode of travel, home trips were the most common, accounting for about two-fifths of the trips in all categories. This is not unexpected since home is the return trip destination of the great majority of trips. Work and business trips ranked second in all population groups, totaling about 28 percent of the trips for all purposes.

In the largest population group, trips for miscellaneous purposes ranked third, social and recreational trips next, and shopping trips last. This same ranking held for auto-

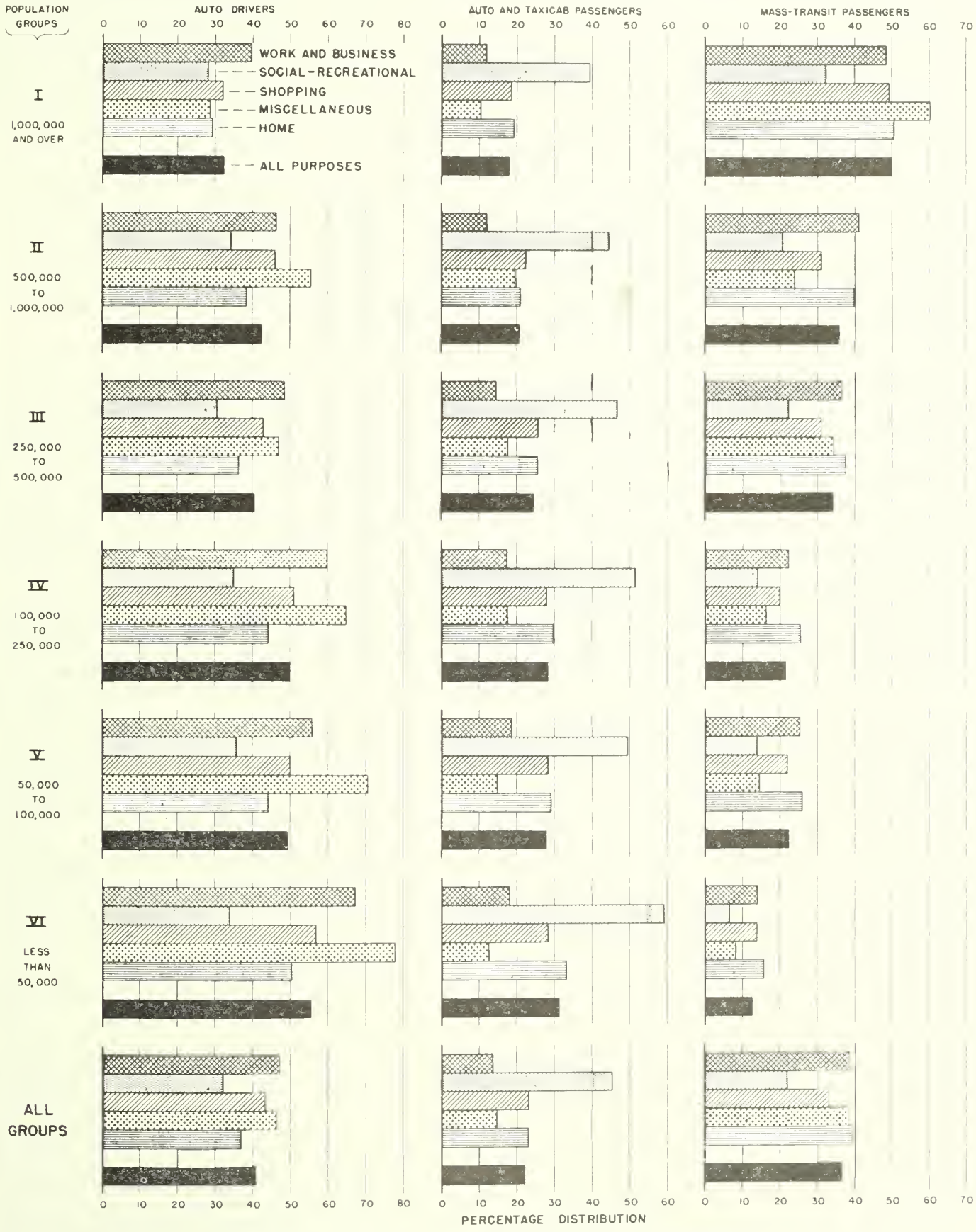

Figure 4.—Percentage distribution of trips according to mode of travel, and further classified by purpose and population group.

Table 7.—Range in percentage of trips for each trip purpose by each mode of travel in six population groups

Mode of travel	Percentage range, by mode of travel, in trips made for purposes of—					
	Work and business	Social and recreation	Shop	Miscellaneous	Home	All purposes
Automobile drivers:						
Maximum	67.4	36.4	57.0	79.0	50.7	55.8
Minimum	39.6	28.2	32.1	28.6	29.4	32.3
Automobile and taxi passengers:						
Maximum	18.4	59.1	28.6	19.9	33.5	31.4
Minimum	11.9	39.4	18.6	10.4	19.4	18.0
Mass-transit passengers:						
Maximum	48.5	32.4	49.3	61.0	51.2	49.7
Minimum	14.3	6.7	14.4	8.2	15.8	12.8

mobile-driver trips in each of the population groups. Although not shown in the tables of this article, a study of the basic data revealed that the relative importance of miscellaneous trips was largely due to the number of automobile drivers who traveled for the purpose of serving passengers. Also in the larger cities a number of change-mode-of-travel trips by mass-transit passengers were classified as miscellaneous. In the other population groups, considering all modes of travel, social and recreational trips ranked third, ahead of trips for miscellaneous purposes.

Among the automobile- and taxi-passenger trips, those for social and recreational purposes ranked second, above work and business trips, and accounted for one-fourth of all trips by this mode. Of the three principal modes, automobile and taxi passengers showed the greatest variation among the different population groups in trips to work and business and for social-recreational purposes, but the least variation in home trips.

Home was the most frequent objective of mass-transit passengers in all population groups. Work and business trips ranked second. For reasons which have been mentioned, miscellaneous trips were relatively important among transit passengers in the largest cities, but in all other population groups social-recreational and shopping trips were about as important as trips for miscellaneous purposes.

Mode distribution

The percentage distribution of trips by mode of travel in the six population groups, shown in table 6, indicates that as the size of city increases the proportion of mass-transit trips generally increases with a corresponding decrease in automobile trips. With some minor exceptions, this trend occurred among trips in each purpose category.

The mode-of-travel pattern by population groups is shown in figure 4. For all purposes combined, the proportion of trips by mass-transit passengers ranged from 50 percent in the cities with over 1 million population to 13 percent in the less than 50,000 population group. On the other hand, trips by automobile drivers ranged from 32 to 56 percent, and automobile and taxi passengers, from 18 to 31 percent. On the basis of individual trip purposes, the ranges among population groups were much greater in some cases, as seen in table 7.

It is evident from figure 4 that the privately owned automobile, considering both drivers and passengers, was the predominant choice for trips to all purposes in cities of less than 1 million population. Automobile travel was also greatly preferred for social and recreational trips by residents of cities in the 1 million or more population group.

Table 8.—Average number of trips per city in each population group classified according to trip purpose by each mode of travel

Population group	Number of cities	Average number (in thousands) of trips per city made for purposes of—					
		Work and business	Social and recreation	Shop	Miscellaneous	Home	Total
Automobile Drivers							
1,000,000 and over	4	286	75	50	104	291	806
500,000–1,000,000	6	185	52	49	73	228	587
250,000–500,000	3	78	24	20	28	81	231
100,000–250,000	20	44	14	14	22	52	146
50,000–100,000	12	20	7	5	10	25	67
Less than 50,000	5	12	5	4	7	14	42
All groups	50	74	22	18	30	84	228
Automobile and Taxi Passengers							
1,000,000 and over	4	86	105	29	38	192	450
500,000–1,000,000	6	48	67	24	26	126	291
250,000–500,000	3	24	37	12	10	57	140
100,000–250,000	20	13	22	7	6	35	83
50,000–100,000	12	7	10	3	2	16	38
Less than 50,000	5	3	8	2	1	10	24
All groups	50	21	30	10	10	53	124
Mass-Transit Passengers							
1,000,000 and over	4	350	86	77	221	507	1,241
500,000–1,000,000	6	164	31	34	32	236	497
250,000–500,000	3	59	17	14	20	84	194
100,000–250,000	20	17	6	5	5	30	63
50,000–100,000	12	9	3	2	2	14	30
Less than 50,000	5	3	1	1	1	4	10
All groups	50	60	15	14	25	90	204
All Modes of Travel							
1,000,000 and over	4	722	266	156	363	990	2,497
500,000–1,000,000	6	397	150	107	131	590	1,375
250,000–500,000	3	161	78	46	58	222	565
100,000–250,000	20	74	42	26	33	117	292
50,000–100,000	12	36	20	10	14	55	135
Less than 50,000	5	18	14	7	9	28	76
All groups	50	155	67	42	65	227	556

Average trips per city

Table 8 contains the number of internal trips made by residents by each mode of travel and for each trip purpose in the average city within each population group. Although the figures are pure arithmetic means of the total trips made in the cities within each population group, the volumes are indicative of what might be expected in other cities of similar size. Of special note is the regularly increasing volume of trips for each trip purpose from the smallest to the largest population group for each mode of travel.

However, there appears to be a near maximum volume of automobile-driver trips for shopping purposes when cities reach the 500,000-1,000,000 population size. In cities of 1 million population and over, trips made by automobile drivers for shopping purposes exceeded those in the 500,000-1,000,000 population group by less than 2 percent. This is reflected in table 6 which shows that automobile drivers made only 32 percent of the shopping trips in the largest cities as compared with 46 percent in the next smaller population group. This difference may be explained partly by the inability of the downtown shopping districts of very large cities to accommodate automobile drivers and partly by the increased availability of transit facilities and taxicabs, particularly around the densely populated areas in the vicinity of the central business district.

Distribution of Trips by Individual Cities

The number of trips by residents according to purpose in each of the 50 urban areas are presented in tables 9 and 10 for automobile drivers, automobile and taxi passengers, mass-transit passengers, and for all modes of travel. In these tables the cities are listed in descending order of population size at the time of the basic survey. The general tendency for a greater volume of trips in the more populous urban areas agrees with the same relationship already mentioned in the discussion of popula-

tion groups, but in the case of individual cities several exceptions are apparent. The more obvious exceptions are readily noticed.

The residents of San Juan made far fewer automobile-driver and automobile- and taxi-passenger trips than persons living in mainland cities of the same size. This relatively small number of trips existed throughout all the major purposes, but applied particularly to shopping trips. The abnormally high number of trips made in Philadelphia for miscellaneous purposes may be related to the large number of mass-transit passenger trips for the intermediate purpose of changing mode of travel. In the St. Paul-Minneapolis area, an unusually large number of trips for social and recreational purposes were made by automobile.

The high volume of mass-transit passenger trips in Philadelphia, St. Louis, and Honolulu is noteworthy, and conversely the relatively small number of automobile-driver and automobile- and taxi-passenger trips in the same cities. In Houston there was an exceptionally large volume of trips by modes other than mass transit, particularly for shopping and miscellaneous purposes. A large number of automobile trips for all purposes is noted in Grand Rapids and Wichita. However, the relative stability of work and business trips and homeward-bound trips is significant throughout all cities.

Purpose distribution

The percentages of trips for each purpose in each of the 50 urban areas are presented in figure 5. Although generally displaying a pattern of uniformity in the percentage distribution of trip purposes within each city, this chart reveals several proportional trip variations which are not readily apparent in the tables of absolute trip volumes.

The large percentage of trips for miscellaneous purposes in Philadelphia again reflects the volume of trips made by mass-transit passengers for the purpose of changing mode of travel. In Wisconsin Rapids the high percentage of miscellaneous trips may be explained by the fact that over 90 percent of the miscellaneous transit trips in this small Wisconsin city were to school. Madison, Wis., a university city, also had a relatively large proportion of trips to school. The percentage of work and business trips is especially high in St. Louis, particularly among automobile and taxi passengers. This is undoubtedly due in part to the time of the survey which was begun just before the end of World War II. There are other extremes of more or less importance, such as the relatively small pro-

Table 9.—Number of trips by automobile drivers and automobile and taxi passengers in each of 50 cities in six population groups, classified according to trip purpose

City	Population group	Number of automobile-driver trips made for purposes of—						Number of automobile- and taxi-passenger trips made for purposes of—					
		Work and business	Social and recreation	Shop	Miscellaneous	Home	Total	Work and business	Social and recreation	Shop	Miscellaneous	Home	Total
Philadelphia, Pa.	1,000,000 and over.	301,490	69,502	32,648	72,655	253,419	729,714	76,711	94,494	23,336	45,475	180,142	420,158
San Francisco, Calif.		423,673	113,942	86,146	208,899	388,421	1,221,081	117,717	163,363	47,283	51,975	260,284	640,622
Newark, N. J.		223,839	56,575	36,654	47,579	278,601	643,248	65,513	66,022	21,021	20,164	154,154	326,874
Washington, D. C.		194,301	60,873	45,886	85,263	145,210	631,533	84,072	96,660	25,047	33,706	174,892	414,377
Total		1,143,303	300,892	201,334	414,396	1,165,651	3,225,576	344,013	420,539	116,687	151,320	769,472	1,802,031
St. Louis, Mo.	500,000–1,000,000.	167,001	31,195	21,747	18,806	216,635	455,384	30,253	12,054	4,526	2,931	45,719	95,483
St. Paul-Minneapolis, Minn.		249,043	85,221	53,954	109,868	285,156	783,242	56,491	128,090	31,694	22,648	178,663	417,586
Baltimore, Md.		138,682	30,395	21,055	22,263	136,185	348,580	44,871	35,016	13,278	18,835	91,386	203,386
Houston, Tex.		283,079	82,573	114,278	161,984	388,703	1,030,617	83,945	112,089	60,187	69,110	242,110	567,441
Dallas, Tex.		148,116	45,630	61,217	89,696	208,231	552,890	40,814	68,188	22,573	33,700	115,969	281,244
Seattle, Wash.		124,257	34,461	24,318	35,910	135,261	354,207	33,336	46,603	12,901	9,224	79,670	181,734
Total		1,110,178	309,475	296,569	438,527	1,370,171	3,524,920	289,710	402,040	145,159	156,448	753,517	1,746,874
Portland, Oreg.	250,000–500,000	143,170	41,536	37,772	55,454	140,772	418,704	34,619	69,522	20,424	9,722	89,206	223,493
Norfolk, Va.		80,240	26,500	19,080	24,220	92,710	242,750	30,960	31,990	12,940	18,180	70,360	164,430
San Juan, P. R.		10,948	4,098	1,389	5,004	9,384	30,823	5,305	8,639	1,411	3,554	12,630	31,539
Total		234,358	72,134	58,241	84,678	242,866	692,277	70,884	110,151	34,775	31,456	172,196	419,462
Wichita, Kans.	100,000–250,000	109,142	26,936	37,051	67,088	131,879	372,096	41,478	32,520	19,857	21,355	84,004	199,214
Grand Rapids, Mich.		89,493	34,859	34,889	39,578	111,094	309,913	22,531	57,674	18,282	9,177	77,351	185,015
Honolulu, T. H.		40,100	29,282	14,118	27,995	63,209	174,704	15,335	43,022	9,644	11,707	64,443	144,151
Sacramento, Calif.		67,802	16,968	21,702	38,760	69,840	215,072	18,328	21,922	10,078	6,801	41,130	98,259
Salt Lake City, Utah		44,212	15,343	8,499	13,073	51,126	132,253	15,728	23,765	5,590	3,173	37,338	85,594
Wilmington, Del.		37,525	11,243	7,275	15,877	38,561	110,481	15,446	17,014	3,815	6,384	33,029	75,688
Phoenix, Ariz.		62,481	21,784	32,252	42,374	78,069	236,960	17,834	31,533	16,397	12,997	49,599	128,360
Tacoma, Wash.		37,948	11,381	8,118	14,595	40,429	112,471	8,843	13,892	3,986	3,294	21,744	51,759
Spokane, Wash.		32,262	9,028	5,964	7,748	42,510	97,512	8,706	10,201	3,974	3,070	22,087	48,038
Scranton, Pa.		17,326	5,278	2,817	4,422	21,926	51,769	5,795	10,249	3,430	962	18,151	38,587
Duluth, Minn., Superior, Wis.		34,449	10,559	7,424	13,089	31,458	96,979	9,230	23,006	5,183	2,582	26,015	66,016
Chester, Pa.		15,279	4,860	4,023	4,843	21,601	50,606	6,694	6,057	3,029	1,243	15,022	32,045
Tucson, Ariz.		45,830	14,340	17,110	23,720	53,510	154,510	11,565	20,942	9,387	8,144	31,832	81,870
Lansing, Mich.		46,857	12,212	14,461	23,259	46,800	143,589	10,893	19,009	5,421	5,192	25,683	66,198
Reading, Pa.		24,896	4,177	2,818	7,993	21,022	60,906	6,579	4,229	1,676	2,002	11,928	26,414
Albuquerque, N. Mex.		38,940	14,575	10,769	19,888	45,287	129,459	11,258	27,045	4,753	4,085	33,137	80,278
Rockford, Ill.		47,261	17,592	11,932	13,891	65,040	155,716	12,988	22,768	8,353	2,179	37,307	83,595
Saginaw, Mich.		40,051	10,614	16,068	25,296	50,860	142,889	11,176	18,900	7,311	4,539	30,063	71,989
Madison, Wis.		31,980	9,943	8,359	20,729	33,235	104,246	8,015	15,314	4,225	6,334	21,128	55,016
Harrisburg, Pa.		25,130	6,057	5,121	9,474	25,130	70,912	6,222	9,757	2,405	1,527	15,804	35,715
Total		888,964	287,031	270,770	433,692	1,042,586	2,923,043	264,644	428,819	146,796	116,747	696,795	1,653,801
Johnstown, Pa.	50,000–100,000	13,488	4,356	4,944	3,984	13,956	40,728	3,061	6,183	2,415	1,166	9,639	22,464
Altoona, Pa.		16,224	6,296	5,423	6,186	20,940	55,069	3,936	9,054	3,512	856	13,811	31,169
Muskegon, Mich.		24,158	12,721	6,969	11,359	32,209	87,416	8,898	21,526	3,975	2,109	25,787	62,295
Pontiac, Mich.		22,089	5,809	5,582	13,195	27,648	74,323	9,690	9,669	3,162	2,603	19,326	44,450
Columbus, Ga.		17,602	5,507	5,882	7,212	25,658	61,861	8,377	4,227	1,928	2,929	13,393	30,854
Racine, Wis.		25,117	8,384	7,303	16,775	27,210	84,789	5,642	11,721	3,385	3,366	15,862	39,976
Macon, Ga.		17,645	1,953	2,549	6,137	21,233	49,517	8,413	3,711	2,214	1,477	14,275	30,090
York, Pa.		33,651	7,777	5,381	13,841	26,755	87,405	9,377	9,080	3,477	2,264	16,999	41,197
Charleston, S. C.		13,227	5,957	3,718	4,222	18,720	45,844	6,203	8,662	2,435	1,071	15,493	33,864
Kalamazoo, Mich.		24,162	10,237	5,329	10,141	29,911	79,780	6,424	11,464	3,223	2,757	17,568	41,436
Bay City, Mich.		21,430	13,660	9,085	16,461	32,834	93,470	6,313	18,884	4,132	3,169	22,988	55,486
Williamsport, Pa.		13,772	5,304	2,908	8,198	17,890	48,072	3,362	5,902	2,083	1,088	10,140	22,575
Total		242,565	87,961	65,073	117,711	294,964	808,274	79,696	120,083	35,941	24,855	195,281	455,856
Fargo, N. Dak., Moorhead, Minn.	Less than 50,000.	22,435	7,795	3,996	12,797	22,502	69,525	6,286	18,315	2,286	1,747	17,950	46,584
Sharon-Farrell, Pa.		12,103	5,267	4,848	7,209	15,508	44,935	3,174	8,364	2,306	698	10,916	25,458
Norristown, Pa.		7,097	3,198	1,754	3,310	11,949	27,308	3,046	2,927	886	801	6,464	14,124
Appleton, Wis.		13,309	4,277	6,058	7,929	14,945	46,518	2,506	6,895	3,056	1,154	8,631	22,242
Wisconsin Rapids, Wis.		5,536	1,912	2,188	4,124	6,776	20,536	1,402	2,249	906	1,320	3,407	9,284
Total		60,480	22,449	18,844	35,369	71,680	208,822	16,414	38,750	9,440	5,720	47,368	117,692
Grand total	All groups	3,679,848	1,079,942	910,831	1,524,373	4,187,918	11,382,912	1,065,361	1,520,382	488,798	486,546	2,634,629	6,195,716

portion of work and business trips in Honolulu, Muskegon, and Bay City, which, in a sense, are somewhat offset by social and recreational trips. However, in spite of these variations among individual cities, the overall effect is to reemphasize the essentially uniform pattern of trip purposes among the population groups.

Though not shown in this article, similar data also were developed for each separate mode of travel. In most cities the combination of work-business and home trips accounted for about 70 percent of all automobile-driver trips as well as mass-transit passenger trips, with social-recreational and shopping trips each accounting for another 7 to 10 percent. On the other hand, trips by automobile and taxi passengers were more frequently made for a social or recreational purpose rather than work or business. Social and recreational trips generally amounted to one-fourth of the total trips by passengers in automobiles and taxis.

Among automobile drivers, trips to home comprised the major portion of the travel in 41 cities. Work and business trips ranked second in these cities and were foremost in the other nine cities.

Homeward-bound trips also ranked first among automobile and taxi passengers in all cities except one. In the Fargo-Moorhead area, social and recreational trips ranked first for this mode of travel. In all but 10 of the remaining 49 cities, social-recreational trips ranked second and were followed by work and business trips. This order was reversed in the remaining 10 cities.

The pattern of trip purposes for mass-transit passengers resembled the automobile-driver pattern more than that of automobile and taxi passengers, but among transit passengers, home trips predominated in all cities without exception. Work and business trips ranked second in all but two cities, Philadelphia and Wisconsin Rapids, where changing-mode-of-travel and school trips caused the miscellaneous group to exceed work and business trips.

The composite of all modes of travel followed the pattern of mass-transit passengers with home trips predominating in all cities, followed by work and business trips in all but Honolulu and Bay City, where social-recreational trips ranked second.

These consistencies in trip patterns suggest the possibility of utilizing the present data in making estimates in cities where surveys have not been completed. Although the ranking of trip purposes is fairly uniform, the limits of the individual percentages show wide variations not directly related to the size of

Table 10.—Number of trips by mass-transit passengers and by all modes of travel in each of 50 cities in six population groups, classified according to trip purpose

City	Population group	Number of mass-transit passenger trips made for purposes of—						Number of trips by all modes of travel for purposes of—					
		Work and business	Social and recreation	Shop	Miscellaneous	Home	Total	Work and business	Social and recreation	Shop	Miscellaneous	Home	Total
Philadelphia, Pa.	1,000,000 and over.	583,557	135,154	126,102	692,382	861,013	2,398,208	961,758	299,150	182,086	810,512	1,294,574	3,548,080
San Francisco, Calif.		296,825	79,652	66,397	86,241	414,667	943,782	838,215	356,957	199,826	347,115	1,063,372	2,805,485
Newark, N. J.		284,434	78,023	78,313	54,244	453,361	948,375	573,786	200,620	135,988	121,987	886,116	1,918,497
Washington, D. C.		237,164	52,699	38,872	51,167	298,058	677,960	515,537	210,232	109,805	170,136	718,160	1,723,870
Total		1,401,980	345,528	309,684	884,034	2,027,099	4,968,325	2,889,296	1,066,959	627,705	1,449,750	3,962,222	9,995,932
St. Louis, Mo.	500,000–1,000,000.	428,806	69,134	70,793	30,495	562,209	1,161,437	626,060	112,383	97,066	52,232	824,563	1,712,304
St. Paul-Minneapolis, Minn.		137,017	29,327	33,728	30,394	201,235	431,701	442,551	242,638	119,376	162,910	665,054	1,632,529
Baltimore, Md.		201,560	44,773	42,922	46,233	307,263	642,751	385,113	110,184	77,255	87,331	534,834	1,194,717
Houston, Tex.		66,022	8,630	14,227	44,125	119,265	252,269	433,046	203,292	188,692	275,219	750,078	1,850,327
Dallas, Tex.		68,066	7,421	12,317	22,299	95,968	206,071	256,996	121,239	96,107	145,695	420,168	1,040,205
Seattle, Wash.		81,675	25,713	27,015	16,532	132,308	283,243	239,268	106,777	64,234	61,666	347,239	819,184
Total		983,146	184,998	201,002	190,078	1,418,248	2,977,472	2,383,034	896,513	642,730	785,053	3,541,936	8,249,266
Portland, Oreg.	250,000–500,000	75,407	27,190	24,837	10,838	110,779	249,051	253,196	138,248	83,033	76,014	340,757	891,248
Norfolk, Va.		40,920	8,230	7,180	6,060	55,350	117,740	152,120	66,720	39,200	48,460	218,420	524,920
San Juan, P. R.		59,449	16,889	10,040	43,450	85,142	214,970	75,702	29,626	12,840	52,008	107,156	277,332
Total		175,776	52,309	42,057	60,348	251,271	581,761	481,018	234,594	135,073	176,482	666,333	1,693,500
Wichita, Kans.	100,000–250,000	18,508	3,484	5,349	8,927	31,441	67,709	169,128	62,940	62,257	97,370	247,324	639,019
Grand Rapids, Mich.		23,407	6,942	5,704	3,331	34,176	73,560	135,431	99,475	58,875	52,086	222,621	568,488
Honolulu, T. H.		34,179	18,845	9,064	17,751	82,366	162,205	89,614	91,149	32,826	57,453	210,018	481,060
Sacramento, Calif.		14,517	3,708	3,917	8,770	27,344	58,256	100,647	42,598	35,697	54,331	138,314	371,587
Salt Lake City, Utah		21,079	7,642	6,332	3,735	35,060	73,848	81,019	46,750	20,421	19,981	123,524	291,695
Wilmington, Del.		28,532	9,317	6,551	7,244	46,242	97,886	81,503	37,574	17,641	29,505	117,832	284,055
Phoenix, Ariz.		15,863	5,046	5,484	10,897	34,028	71,318	96,178	58,363	54,133	66,268	161,696	436,638
Tacoma, Wash.		14,967	4,724	4,742	5,340	25,400	55,173	61,758	29,997	16,846	23,229	87,573	219,403
Spokane, Wash.		14,915	4,979	6,177	5,057	32,090	63,218	55,883	24,208	16,115	15,875	96,687	208,768
Scranton, Pa.		15,765	9,282	14,017	1,960	40,238	81,262	38,886	24,809	20,264	7,344	80,315	171,618
Duluth, Minn., Superior, Wis.		17,648	7,244	4,953	2,468	27,658	59,971	61,327	40,809	17,560	18,139	85,131	222,966
Chester, Pa.		8,197	1,922	2,311	3,057	12,680	28,167	30,170	12,839	9,363	9,143	49,303	110,818
Tucson, Ariz.		7,310	2,490	2,640	5,590	16,290	34,320	64,705	37,772	29,137	37,454	101,632	270,700
Lansing, Mich.		8,543	2,719	2,583	3,797	14,478	32,120	66,293	33,940	22,465	32,248	86,961	241,907
Reading, Pa.		23,717	4,907	4,731	3,552	32,885	69,792	55,192	13,313	9,225	13,547	65,835	157,112
Albuquerque, N. Mex.		8,668	3,597	3,069	2,266	15,290	32,890	58,866	45,217	18,591	26,239	93,714	242,627
Rockford, Ill.		10,911	3,059	6,749	1,354	20,484	42,557	71,160	43,419	27,034	17,424	122,831	281,868
Saginaw, Mich.		6,217	2,074	1,427	871	9,471	20,060	57,444	31,588	24,806	30,706	90,394	234,938
Madison, Wis.		10,495	4,008	2,500	8,785	20,051	45,839	50,490	29,265	15,084	35,848	74,414	205,101
Harrisburg, Pa.		26,782	9,578	5,861	3,299	38,592	84,112	58,134	25,392	13,387	14,300	79,526	190,739
Total		330,220	115,567	104,161	108,051	596,264	1,254,263	1,483,828	831,417	521,727	658,490	2,335,645	5,831,107
Johnstown, Pa.	50,000–100,000	11,424	3,924	3,132	1,176	17,940	37,596	27,973	14,463	10,491	6,326	41,535	100,788
Altoona, Pa.		6,107	1,823	2,391	946	10,300	21,567	26,267	17,173	11,326	7,988	45,051	107,805
Muskegon, Mich.		6,373	4,022	2,307	574	11,360	24,636	39,429	38,269	13,251	14,042	69,356	174,347
Pontiac, Mich.		7,122	1,949	1,648	4,813	12,768	28,300	38,901	17,427	10,392	20,611	59,742	147,073
Columbus, Ga.		21,645	2,894	3,765	4,024	30,903	63,231	47,624	12,628	11,575	14,165	69,954	155,946
Racine, Wis.		7,653	2,916	2,390	2,832	11,257	27,048	38,412	23,021	13,078	22,973	54,329	151,813
Macon, Ga.		20,549	4,418	5,006	2,915	33,154	66,042	46,607	10,082	9,769	10,529	68,662	145,649
York, Pa.		4,281	1,377	1,496	1,345	6,537	15,036	47,309	18,234	10,354	17,450	50,291	143,638
Charleston, S. C.		10,498	5,068	2,261	789	15,885	34,501	29,928	19,687	8,414	6,082	50,098	114,209
Kalamazoo, Mich.		7,189	2,546	2,247	3,031	11,144	26,157	37,775	24,247	10,799	15,929	58,623	147,373
Bay City, Mich.		4,955	1,978	1,239	1,337	7,017	16,526	32,698	34,522	14,456	20,967	62,839	165,482
Williamsport, Pa.		2,401	781	896	498	3,971	8,547	19,535	11,987	5,887	9,784	32,001	79,194
Total		110,197	33,696	28,778	24,280	172,236	369,187	432,458	241,740	129,792	166,846	662,481	1,633,317
Fargo, N. Dak., Moorhead, Minn.	Less than 50,000.	3,543	1,690	1,310	885	5,994	13,422	32,264	27,800	7,592	15,429	46,446	129,531
Sharon-Farrell, Pa.		4,229	1,489	2,021	585	7,688	16,012	19,506	15,120	9,175	8,492	34,112	86,405
Norristown, Pa.		3,908	842	973	1,759	6,527	14,009	14,051	6,967	3,613	5,870	24,940	55,441
Appleton, Wis.		1,052	356	445	325	1,994	4,172	16,867	11,528	9,559	9,408	25,570	72,932
Wisconsin Rapids, Wis.		52	12	4	116	220	404	6,990	4,173	3,098	5,560	10,403	30,224
Total		12,784	4,389	4,753	3,670	22,423	48,019	89,678	65,588	33,037	44,759	141,471	374,533
Grand total	All groups	3,014,103	736,487	690,435	1,270,461	4,487,541	10,199,027	7,759,312	3,336,811	2,090,064	3,281,380	11,310,088	27,777,655

city and indicate that such a basis would provide only a rude forecast at best.

The ranges in percentages of trips for each trip purpose by each mode of travel are shown in table 11. Despite the wide range between the maximum and minimum percentages, it is seen later that for any particular urban area it is possible to make a fairly reasonable forecast of the absolute volume of trips from which percentages may be computed.

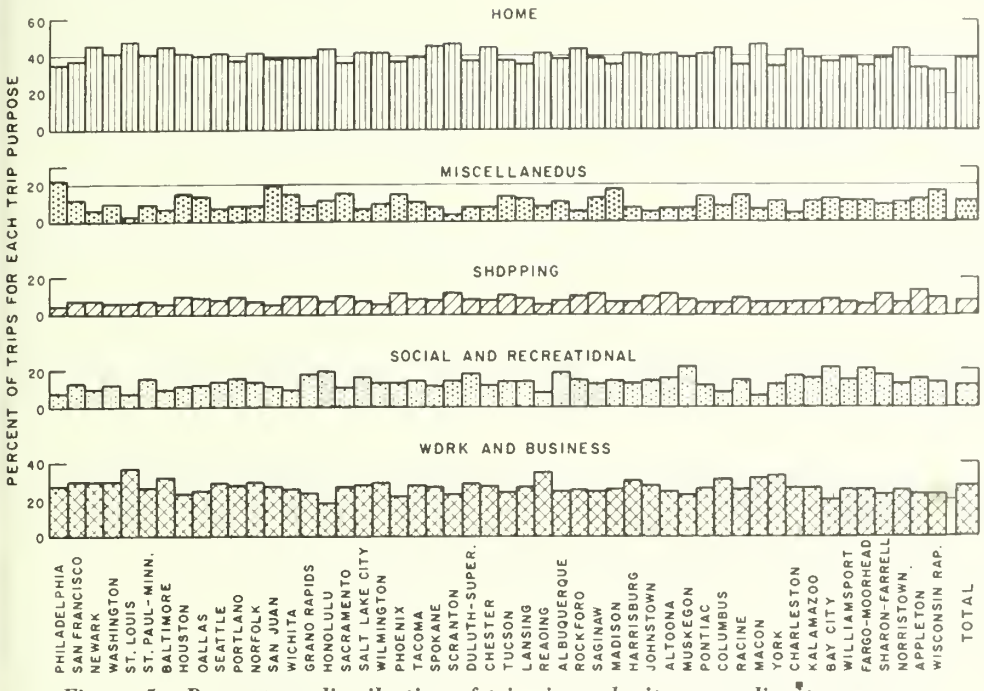

Figure 5.—Percentage distribution of trips in each city, according to purpose.

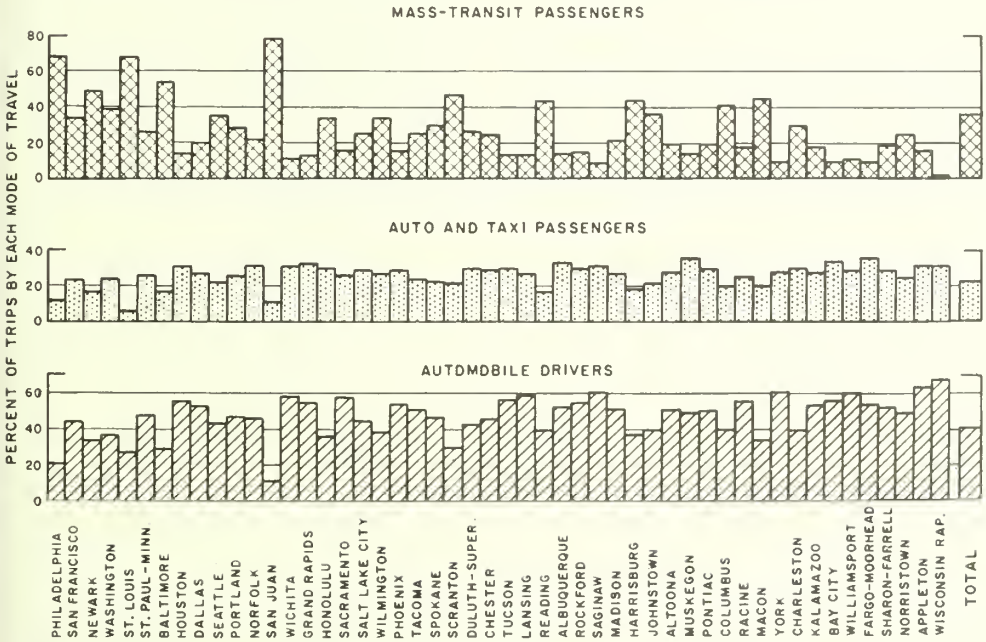

Figure 6.—Percentage distribution of trips in each city, according to mode of travel.

Table 11.—Range in percentage of trips for each trip purpose by each mode of travel in 50 cities

Mode of travel	Percentage range, by mode of travel, in trips made for purposes of—				
	Work and business	Social and recreation	Shop	Miscellaneous	Home
Automobile drivers:					
Maximum	41.3	16.8	13.6	20.1	47.6
Minimum	22.9	4.0	4.5	4.1	30.4
Automobile and taxi passengers:					
Maximum	31.7	39.3	13.7	12.2	47.9
Minimum	10.6	12.3	4.5	2.5	36.7
Mass-transit passengers:					
Maximum	36.9	16.3	17.3	28.9	54.5
Minimum	12.9	3.0	1.0	2.3	35.9
All modes of travel:					
Maximum	36.6	22.0	13.1	22.8	48.2
Minimum	18.6	6.9	4.6	3.1	34.4

Mode distribution

The percentage distribution of trips in the 50 individual cities by mode of travel is presented in figure 6. The most noticeable difference from the previous distribution by trip purpose is the relative lack of uniformity among the several cities when considering travel mode. While not included in this article, similar percentages were developed for each trip purpose and a variable pattern was found in each case. The ranges in the percentage of trips by each mode of travel for each trip purpose and for all purposes are shown in table 12.

Besides being small in absolute volumes, trips by automobile drivers and automobile and taxi passengers were also few on a relative basis in San Juan, where 7 out of 9 persons making trips traveled as mass-transit passengers, largely in "publicos" (privately owned public conveyances, usually station wagons, which generally operate over established routes but with no fixed schedule). On the other hand, exceptionally high percentages of automobile trips were observed for each trip purpose in cities of Texas, New Mexico, Arizona, California, Washington, Michigan, and Wisconsin.

It may be that these variations are related to the period during which the basic studies were made or to the geographical area in which the cities are located. Some of the studies where mass-transit facilities played an important role were made during or shortly after World War II when automobile driving was restricted. Also, other evidence indicates that the preference for automobile travel has increased progressively over the decade during which the studies were made in the various cities. Insofar as location is concerned, it is not unusual to find a particularly high proportion of automobile-driver trips in the Southwestern and Pacific States, and certain States in the Great Lakes region where automobile ownership and travel are relatively high.

Table 12.—Range in percentage of trips by each mode of travel for each trip purpose in 50 cities

Purpose of trip	Percentage range, by purpose of trip, in trips made by—		
	Automobile drivers	Automobile and taxi passengers	Mass-transit passengers
Work and business:			
Maximum	79.2	21.9	78.5
Minimum	14.5	4.8	.7
Social and recreation:			
Maximum	45.9	65.9	61.5
Minimum	13.8	10.7	.3
Shop:			
Maximum	70.6	35.4	78.2
Minimum	10.8	4.7	.1
Miscellaneous:			
Maximum	84.9	37.5	85.4
Minimum	9.0	5.6	2.1
Home:			
Maximum	65.1	38.7	79.5
Minimum	8.8	5.6	2.1
All purposes:			
Maximum	67.9	35.7	77.5
Minimum	11.1	5.6	1.3

In spite of the noticeable lack of uniformity as far as mode of travel for each trip purpose is concerned, there was an overall trend for a larger percentage of automobile trips in smaller cities as would be expected. Conversely, there seemed to be a general trend toward a larger percentage of mass-transit passenger trips in the larger cities. Mass transit was the predominant mode of travel in the largest cities, but automobile drivers comprised over half of the vehicular trips by residents in most of the medium-size and smaller cities. These trends appeared among trips for each purpose.

Trips from Purpose to Purpose

All of the previous discussion has dealt with the purpose of trips in connection with their point of destination. This section considers the purpose from which the trips were made at points of origin, as related to the destination purpose. This type of information is presented only in summary form for all 50 urban areas, although detailed data are available from individual city reports. The number of trips made by persons "from" a purpose "to" a purpose are included. This somewhat unusual phraseology is used to express an idea that could not otherwise be expressed precisely in so few words. It describes not only why a person made a trip to his destination, but why he was at the place he left.

Table 13 shows the volume of trips in all 50 urban areas from each purpose to each purpose for each mode of travel. The predominant purposes of trips by all modes of travel were from home to work and business, followed closely by trips from work and business to home. These same trips were dominant among mass-transit passengers and automobile drivers, but ranked second among automobile and taxi passengers. The trips from work or business to home did not quite equal the volume of trips in the reverse direction because of the intermediate trips from work or business for some other purpose prior to returning home. For instance, some of this difference was accounted for by the excess of trips from social-recreational purposes to home, over and above the number of trips from home for social and recreational purposes. Also pedestrian trips, not included in the basic surveys, could have accounted for some of the apparent discrepancies.

Trips between home and social-recreational activities were the next most important category (after the home and work-business cycle) among the trips by all modes of travel combined, but they were the most important purpose-to-purpose category among automobile and taxi passengers. Trips between home and miscellaneous purposes ranked second for automobile drivers and mass-transit passengers, third for all modes of travel combined, and fourth for automobile and taxi passengers. The third ranking category among automobile drivers and mass-transit passengers was home trips to and from social-recreational purposes. Trips between home and shopping ranked third with automobile and taxi passengers and fourth with each of the other modes of travel and with all modes combined. The only other significant purpose-to-purpose categories were the automobile- and taxi-passenger trips from one social or recreational purpose to another, trips from work or business to work or business by automobile drivers, and trips between work or business and miscellaneous purposes by each mode of travel.

Table 13 also shows the percentage distribution of trips from each purpose to each purpose for all travel modes. Trips from home to work and business by mass-transit passengers were the foremost type of internal trips by residents of the 50 urban areas. These trips accounted for nearly 10 percent of the total trips by all modes for all purposes. Trips either to or from home were the most numerous of all. The only other categories of trips accounting for 1 percent or more of the total were trips by automobile drivers for work or business and miscellaneous purposes, and social-recreational trips by automobile and taxi passengers.

Table 13 is the basis for figure 7 which presents the percentage distribution of trips from each purpose to each purpose, and that proportion attributable to each mode of travel. Since trips are grouped first by trip purpose and then by all purposes, each trip is represented at least twice in this chart. The arrows indicate the direction of trip purpose. In the upper left-hand corner of the chart, for instance, under the home category, it may be

Table 13.—Number and percentage of trips by each mode of travel in 50 cities from each purpose to each purpose

Trips from—	Trips to—											
	Work and business		Social and recreation		Shop		Miscellaneous		Home		Total	
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
AUTOMOBILE DRIVERS												
Work and business	1,137,747	4.1	69,070	0.2	103,456	0.4	338,619	1.2	2,042,942	7.4	3,691,834	13.3
Social and recreation	31,001	.1	135,366	.5	43,584	.2	93,036	.3	774,933	2.8	1,077,920	3.9
Shop	43,624	.2	46,590	.2	138,549	.5	43,602	.2	628,494	2.2	900,759	3.3
Miscellaneous	340,199	1.2	93,187	.3	74,324	.2	237,668	.9	741,549	2.7	1,486,927	5.3
Home	2,127,277	7.6	735,729	2.7	551,018	2.0	811,448	2.9	----------	------	4,225,472	15.2
Total	3,679,848	13.2	1,079,942	3.9	910,831	3.3	1,524,373	5.5	4,187,918	15.1	11,382,912	41.0
AUTOMOBILE AND TAXI PASSENGERS												
Work and business	117,942	0.4	42,639	0.2	36,541	0.1	61,531	0.2	837,854	3.0	1,096,507	3.9
Social and recreation	21,597	.1	305,659	1.1	40,438	.1	38,734	.1	1,159,437	4.2	1,565,865	5.6
Shop	12,575	.1	41,213	.1	73,229	.2	9,402	------	351,404	1.3	487,823	1.7
Miscellaneous	48,862	.2	51,781	.2	17,394	.1	26,310	.1	285,934	1.0	430,281	1.6
Home	864,385	3.1	1,079,090	3.9	321,196	1.1	350,569	1.3	----------	------	2,615,240	9.4
Total	1,065,361	3.9	1,520,382	5.5	488,798	1.6	486,546	1.7	2,634,629	9.5	6,195,716	22.2
MASS-TRANSIT PASSENGERS												
Work and business	118,402	0.4	35,234	0.1	38,464	0.2	233,567	0.9	2,533,978	9.1	2,959,645	10.7
Social and recreation	9,086	------	35,688	.1	12,208	------	39,639	.2	598,636	2.2	695,257	2.5
Shop	13,399	------	21,087	.1	23,085	.1	44,347	.2	590,706	2.1	692,624	2.5
Miscellaneous	237,287	.9	52,453	.2	53,753	.2	189,914	.6	764,221	2.8	1,297,628	4.7
Home	2,635,929	9.5	592,025	2.1	562,925	2.1	762,994	2.7	----------	------	4,553,873	16.4
Total	3,014,103	10.8	736,487	2.6	690,435	2.6	1,270,461	4.6	4,487,541	16.2	10,199,027	36.8
ALL MODES OF TRAVEL												
Work and business	1,374,091	4.9	146,943	0.5	178,461	0.7	633,717	2.3	5,414,774	19.5	7,747,986	27.9
Social and recreation	61,684	.2	476,713	1.7	96,230	.3	171,409	.6	2,533,006	9.2	3,339,042	12.0
Shop	69,598	.3	108,890	.4	234,763	.8	97,351	.4	1,570,604	5.6	2,081,206	7.5
Miscellaneous	626,348	2.3	197,421	.7	145,471	.5	453,892	1.6	1,791,704	6.5	3,214,836	11.6
Home	5,627,591	20.2	2,406,844	8.7	1,435,139	5.2	1,925,011	6.9	----------	------	11,394,585	41.0
Total	7,759,312	27.9	3,336,811	12.0	2,090,064	7.5	3,281,380	11.8	11,310,088	40.8	27,777,655	100.0

seen that trips in connection with work and business accounted for the largest proportion of home trips. Trips from home to work and business slightly exceeded those in the reverse direction (20.3 percent as compared with 19.5 percent). Mass-transit passengers ranked first in these trips, and automobile and taxi passengers ranked third behind automobile drivers.

Home trips that were linked with social and recreational purposes were fewer than those involving work and business. Their pattern differed from the latter in that trips from home to social-recreational activities were fewer than the reverse trips. Also, in this case, automobile- and taxi-passenger trips were the most numerous, and were followed by automobile-driver and mass-transit passenger trips. As a matter of fact, home trips linked with work and business were made less often by automobile and taxi passengers than home trips linked with a social-recreational purpose. Figure 7 is adaptable similarly to an analysis of trips associated with other or with all to and from purposes.

Table 14 shows the percentage of trips made both to and from each purpose for each mode of travel. Since for each single trip there are two purposes, one from and one to, the totals add to 200 percent. This table formed the basis for figure 8, from which it is apparent that first home and then work and business were the top-ranking purposes among all modes except one. Automobile and taxi passengers traveled more frequently from or to a social-recreational purpose (50 percent) than a work or business purpose (35 percent). Work and business trips were relatively more significant among the automobile drivers, since 65 percent of their trips were for that purpose. Mass-transit passengers were the group most likely to be traveling from or to home. The fact that this purpose accounted for almost 89 percent of their trips may be related to the greater possibility that intermediate trips by these persons were made by walking than in the case of automobile drivers and automobile and taxi passengers. Miscellaneous trips accounted for about

Table 14.—Percentage of trips for each mode of travel in 50 cities, classified according to purpose at both origin and destination

Purpose	Mode of travel [1]			
	Automobile drivers	Automobile and taxi passengers	Mass-transit passengers	All modes of travel
Home	73.9	84.7	88.6	81.8
Work and business	64.8	34.9	58.6	55.8
Social-recreation	19.0	49.8	14.0	24.0
Miscellaneous	26.4	14.8	25.2	23.4
Shopping	15.9	15.8	13.6	15.0
Total	200.0	200.0	200.0	200.0

[1] Percentages add to 200 for each mode of travel because the purpose of each trip is considered twice, at place of origin (purpose from) and at place of destination (purpose to).

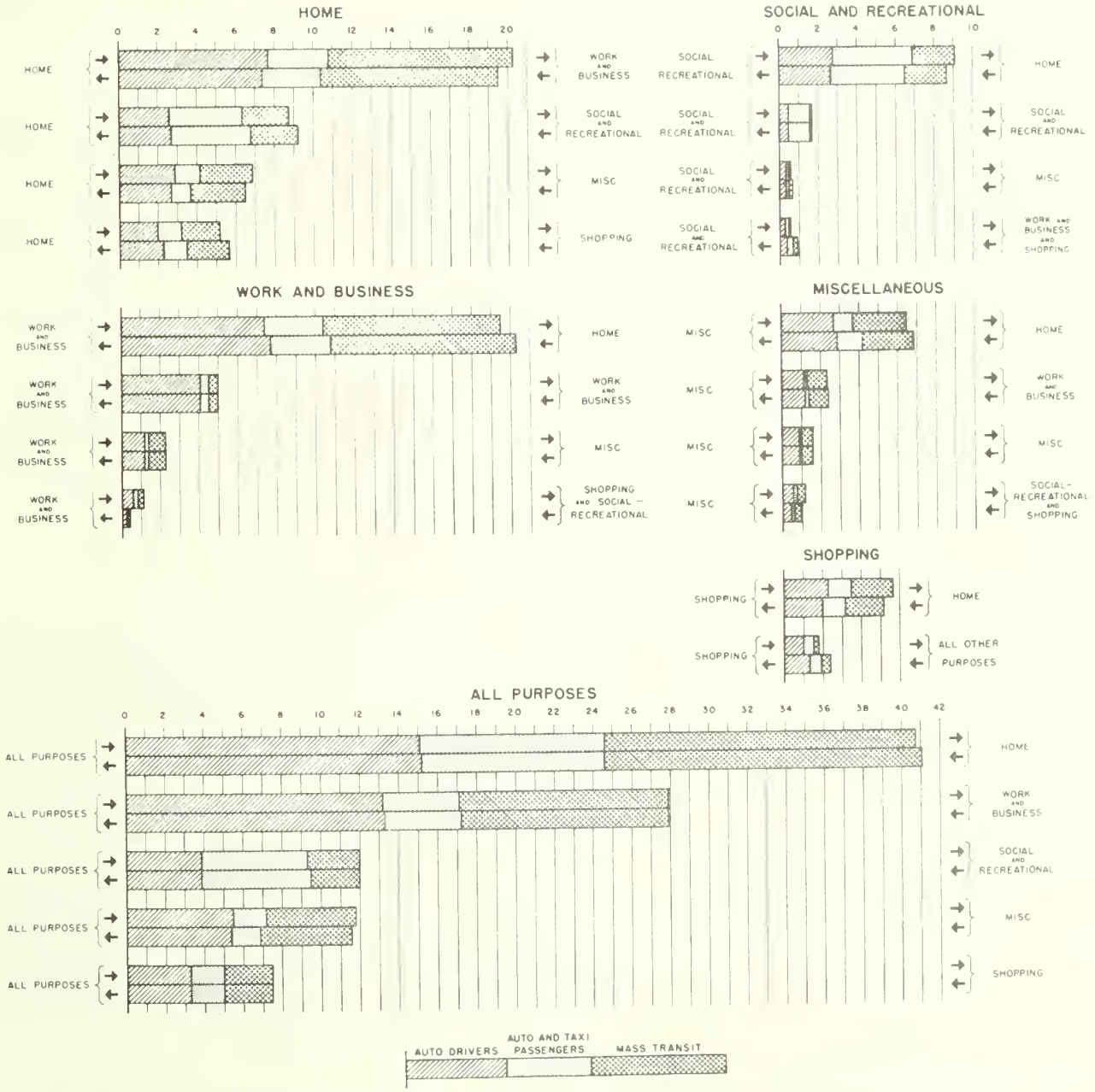

Figure 7.—Percentage distribution of trips from each purpose to each purpose, by mode of travel.

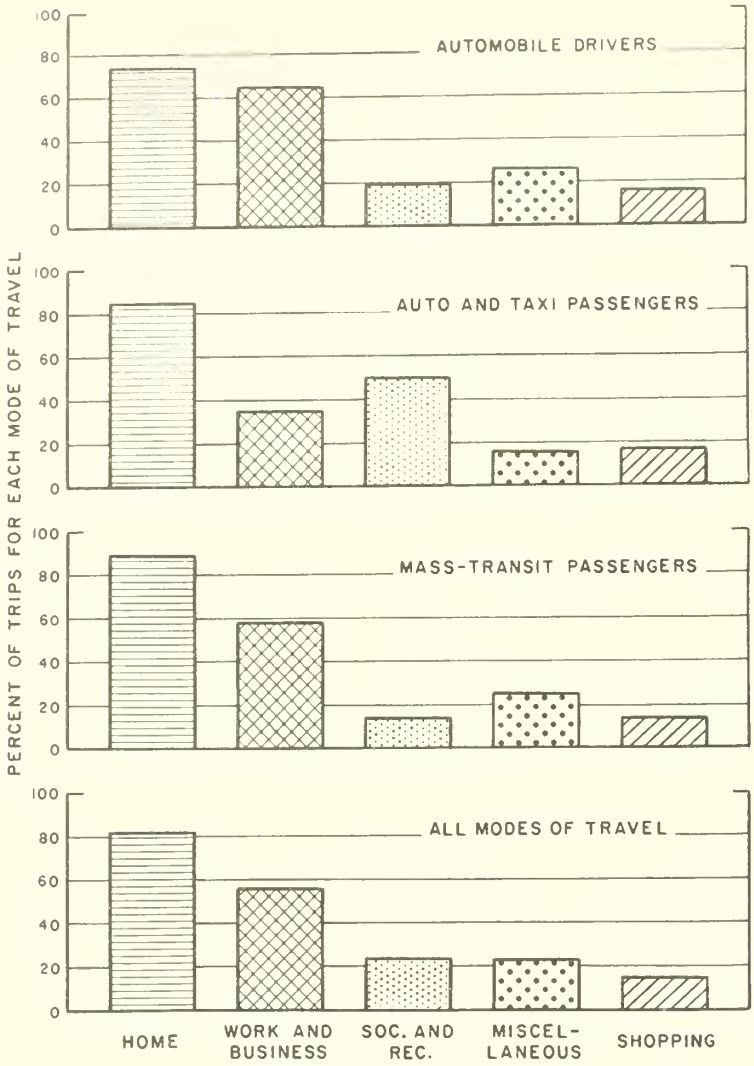

Figure 8.—Percentage distribution of trips, both from and to each purpose, by mode of travel.

one-fourth of the trips by both mass-transit passengers and by automobile drivers. Trips to or from shopping amounted to approximately 15 percent of the trips by each mode of travel.

The percentage distribution of trips from each purpose to each purpose is presented in figure 9 for all modes of travel combined. This chart was constructed in a manner similar to figure 2. It shows, for instance, that trips from home to work and business predominated, accounting for almost 50 percent of the trips from home and over 20 percent of all trips. The reverse trips from work and business to home also accounted for about one-fifth of all trips, but they comprised 70 percent of the trips from work and business. Trips to home accounted for three-fourths of the trips from social-recreational and from shopping purposes, but in comparison with total trips, they represented only 9 and 6 percent, respectively. The large proportion of trips both to and from home, 82 percent, is particularly apparent in figure 9.

Household Characteristics

In addition to data concerning the daily trips of residents, the basic origin and destination surveys of the home-interview type provided information concerning the numbers of dwelling units, automobiles owned, residents, and persons 5 years of age and older. Some of these household characteristics for the 50 urban areas are recorded in table 15.

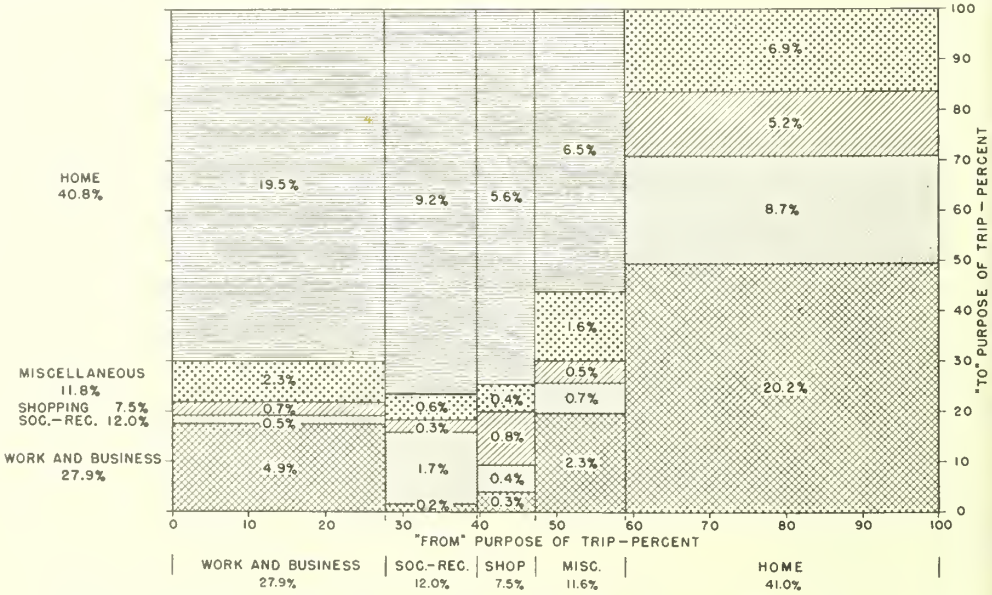

Figure 9.—Percentage distribution of trips from each purpose to each purpose.

By and large they varied directly with population. This pattern is more apparent in table 16, which compares the mean averages for each of six population groups. In this table and in all of the following analyses San Juan, Puerto Rico, was omitted because of the significant differences from the pattern of travel in the continental United States.

Trips Related to Household Characteristics

The ratios of trips by each travel mode to household characteristics are shown in table 17 for the average city in each population group. The ratios of total trips and automobile trips tended to vary inversely with population, while mass-transit trip ratios varied directly with population, as seen in figure 10. The sharp upturn in the pattern for total trips per automobile owned in the highest population group was due to the relatively low automobile ownership ratios in cities of the 1 million or more population group and the greater incidence of mass-transit trips in these cities. The reverse situation caused the low point in this pattern for cities of less than 50,000 population. Some of the other variations of the patterns in the 500,000 to 1,000,000 and 50,000 to 100,000 population groups would be smoothed out by eliminating surveys conducted during World War II.

The basic tables 9 and 10, giving trip purpose and mode of travel in each of the 50 cities, and the household characteristics shown in table 15 may be used to develop similar individual city ratios for each mode of travel and each trip purpose. For each trip purpose there appears to be an inverse linear correlation between population and trips per dwelling unit or trips per person for the automobile travel modes; that is, the larger cities have smaller trip ratios. In the case of trips by mass-transit passengers, the correlations generally appear to be direct for each trip purpose.

The relations existing between a few of these trip ratios and the number of automobiles

Table 15.—Selected household characteristics in each of 50 cities in 6 population groups

City	Population group	Number of dwelling units	Number of passenger cars owned	Number of persons, all ages	Number of persons, 5 years of age and older
Philadelphia, Pa.	1,000,000 and over	659,165	257,907	2,233,531	2,048,388
San Francisco, Calif.		554,200	317,400	1,468,933	1,348,835
Newark, N. J.		436,886	245,151	1,456,947	1,345,138
Washington, D. C.		336,181	203,464	1,109,860	992,644
Total		1,986,432	1,023,922	6,269,271	5,735,005
St. Louis, Mo.	500,000–1,000,000	294,757	143,415	974,545	878,377
St. Paul-Minneapolis, Minn.		299,510	226,815	915,960	825,625
Baltimore, Md.		275,778	123,998	912,809	830,909
Houston, Tex.		272,722	256,300	878,629	765,942
Dallas, Tex.		168,066	153,777	533,606	471,064
Seattle, Wash.		188,732	118,622	518,563	471,911
Total		1,499,565	1,022,927	4,734,112	4,243,828
Portland, Oreg.	250,000–500,000	152,586	103,245	453,128	412,358
Norfolk, Va.		108,000	61,480	335,910	293,270
San Juan, P. R.		63,131	8,011	312,069	267,726
Total		323,717	172,736	1,101,107	973,354
Wichita, Kans.	100,000–250,000	79,534	75,888	238,302	206,529
Grand Rapids, Mich.		65,170	52,795	220,977	199,209
Honolulu, T. H.		51,422	32,692	214,236	184,141
Sacramento, Calif.		79,100	53,900	201,345	179,778
Salt Lake City, Utah		57,103	38,851	196,571	172,557
Wilmington, Del.		49,903	30,190	181,445	162,503
Phoenix, Ariz.		48,221	36,372	161,567	145,198
Tacoma, Wash.		48,008	35,175	138,700	125,002
Spokane, Wash.		48,517	29,644	138,381	124,952
Scranton, Pa.		41,362	22,093	137,089	126,541
Duluth, Minn., Superior, Wis.		42,550	25,596	130,847	119,056
Chester, Pa.		35,206	24,449	127,408	114,709
Tucson, Ariz.		38,690	32,910	126,900	113,730
Lansing, Mich.		35,821	30,252	122,776	110,269
Reading, Pa.		37,910	17,184	119,850	112,504
Albuquerque, N. Mex.		34,884	27,469	116,056	100,817
Rockford, Ill.		36,200	33,100	116,000	102,500
Saginaw, Mich.		31,915	27,028	112,902	101,438
Madison, Wis.		33,365	25,328	104,074	94,300
Harrisburg, Pa.		31,599	16,363	103,303	96,100
Total		926,480	667,279	3,008,729	2,691,833
Johnstown, Pa.	50,000–100,000	23,130	13,828	87,509	80,351
Altoona, Pa.		24,060	16,758	85,347	77,477
Muskegon, Mich.		23,507	18,941	83,724	75,099
Pontiac, Mich.		22,251	17,808	79,431	71,851
Columbus, Ga.		20,307	8,808	79,192	70,621
Racine, Wis.		23,280	18,483	78,033	69,508
Macon, Ga.		20,089	9,529	77,665	69,966
York, Pa.		25,310	20,473	77,350	69,387
Charleston, S. C.		20,258	7,179	73,205	65,390
Kalamazoo, Mich.		22,645	17,198	72,024	65,945
Bay City, Mich.		19,561	15,927	69,231	61,454
Williamsport, Pa.		17,016	14,715	55,216	48,675
Total		261,414	179,647	917,927	825,724
Fargo, N. Dak., Moorhead, Minn.	Less than 50,000	15,617	12,688	49,852	44,030
Sharon-Farrell, Pa.		13,657	9,442	48,432	44,310
Norristown, Pa.		10,282	7,466	39,485	36,106
Appleton, Wis.		11,769	11,073	39,172	33,923
Wisconsin Rapids, Wis.		4,700	4,660	16,504	14,428
Total		56,025	45,329	193,445	172,797
Grand total		5,053,633	3,111,840	16,224,591	14,642,541

per dwelling unit are shown in figure 11. It is noted that in areas of high automobile-ownership ratios, the total trips per person and the automobile trips per person were greater. Also, since automobile-driver trips per automobile tended to increase as automobile ownership increased, the number of trips per vehicle may be expected to increase as the ownership ratio of automobiles per family continues to grow. Whether mileage traveled per vehicle follows the same trend depends upon trip lengths. As in the case of figure 10, these curves are also affected by the data from older studies and by the economic as well as the population characteristics of the cities studied.

Volume of trips and percentage of trips by individual purposes and modes of travel were associated with the ratios of automobiles per dwelling unit and persons per automobile. Although there was fairly good linear correlation between percentage of trips (by purpose and mode) and automobiles per dwelling unit, these correlations were not as high as others relating trips to the absolute household data in each urban area. In the latter case, better correlations were found between volume of trips (by purpose and mode) and the numbers of persons over 5 years of age, automobiles or dwelling units, than between percentage of trips (for a particular purpose or mode of travel) and any one of these variables.

The household characteristic which was most closely related to volume of trips varied, depending upon the mode of travel or purpose of trip. These relations are shown in table 18, together with their respective correlation coefficients. These two-variable, linear correlations were deemed to be sufficiently high to forgo the need for testing correlations based upon second-degree equations or logarithms. However, for convenience of presentation the related scatter diagrams shown in figures 12–16 have been plotted on logarithmic scales.

No attempt was made to associate all household characteristics with the volume of automobile- and taxi-passenger trips, but the scatter diagram in figure 17 suggests that the number of automobiles owned in the area is a good factor.

The relatively low correlation for trips with miscellaneous purposes is not unusual because of the varying nature of such trips. A better correlation factor is hardly required, however, since there is less cause for estimating these miscellaneous trips due to their relatively small number—less than 12 percent of the total. More favorable multiple correlations might be developed if required. For instance, the addition of the factor automobiles owned to the number of persons over 5 years of age raised the correlation with mass-transit passengers from +0.941 to +0.987.

In view of the large number of automobile-driver trips made for the purpose of going to work and for transacting business, these particular trips were also associated with the several household factors. Although total work and business trips in an area were more closely related to dwelling units (a higher correlation coefficient) than total automobile-driver trips were related to automobiles owned, it was found that work and business trips made by automobile drivers were more closely associated to automobiles owned. In the latter comparison, which is illustrated in figure 18, the correlation coefficient was +0.984.

In order to more precisely estimate the volume of trips by each mode of travel for each individual purpose, it would be necessary to determine by means of correlation techniques similar maximum coefficients for the other modes and purposes. Of course, any application of estimates must be consistent with the resulting standard error. Further development is not attempted here, since this article is primarily concerned with existing conditions within the 50 urban areas. However, this discussion should be sufficiently indicative of the types of analysis which may be continued and expanded in an effort to

Table 16.—Average number of dwelling units, passenger cars owned, and residents per city for each of six population groups

Population group	Number of cities	Average number (in thousands) of—			
		Dwelling units	Passenger cars owned	Persons, all ages	Persons, 5 years of age and older
1,000,000 and over	4	497	256	1,567	1,434
500,000–1,000,000	6	250	170	789	707
250,000–500,000	2	130	82	395	353
100,000–250,000	20	46	33	150	135
50,000–100,000	12	22	15	76	69
Less than 50,000	5	11	9	39	35
All groups	49	102	64	325	294

Table 17.—Average ratios per city between number of trips by each mode of travel and selected household characteristics in six population groups

Population group	Trips per dwelling unit by mode of travel				Trips per automobile owned by mode of travel				Trips per person by mode of travel			
	Automobile driver	Automobile and taxi passenger	Mass-transit passenger	Total	Automobile driver	Automobile and taxi passenger	Mass-transit passenger	Total	Automobile driver	Automobile and taxi passenger	Mass-transit passenger	Total
1,000,000 and over	1.62	0.91	2.50	5.03	3.15	1.76	4.85	9.76	0.51	0.29	0.79	1.59
500,000–1,000,000	2.35	1.16	1.99	5.50	3.45	1.70	2.91	8.06	.74	.37	.63	1.74
250,000–500,000	2.54	1.49	1.41	5.44	4.02	2.35	2.22	8.59	.84	.49	.46	1.79
100,000–250,000	3.15	1.79	1.35	6.29	4.38	2.48	1.88	8.74	.97	.55	.42	1.94
50,000–100,000	3.09	1.75	1.41	6.25	4.50	2.54	2.05	9.09	.88	.50	.40	1.78
Less than 50,000	3.73	2.10	.85	6.60	4.61	2.60	1.06	8.27	1.08	.61	.25	1.94

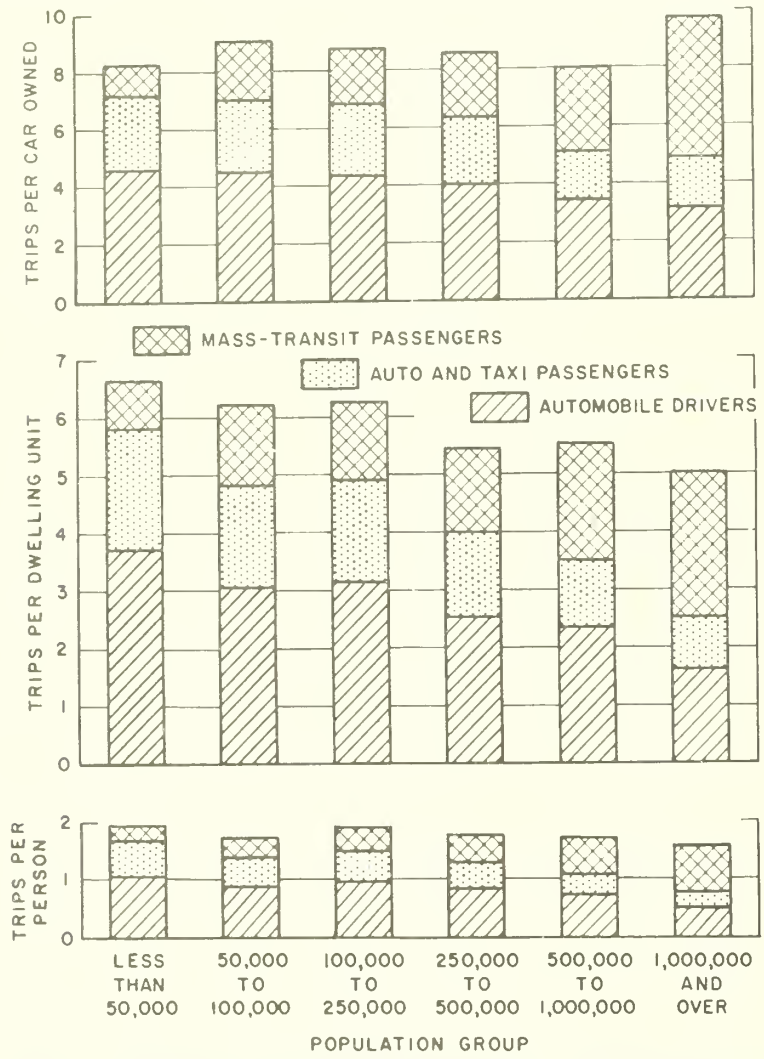

Figure 10.—Relation of trips per person, trips per dwelling unit, and trips per automobile to population size of cities.

develop predictive factors representative of local travel in typical urban areas.

Appendix

Up to this point, the discussion has dealt with several aspects of the travel pattern in 50 urban areas with regard to the 5 major trip purposes and the 3 most important modes of travel. It was mentioned, however, that the basic origin and destination surveys, which provided the data for these analyses, included information with respect to 7 possible travel modes and 10 trip purposes; and in certain cases some rather interesting and significant facets of the total urban travel complex were obscured as a result of the combining processes. Several of the more notable individual aspects are included here.

For all cities, medical-dental trips accounted for 5.5 percent of the trips by taxi passengers, and, conversely, taxi-passenger trips accounted for 4.2 percent of the trips for medical or dental purposes. Changing of mode accounted for 10 percent of the train-passenger trips and train passengers accounted for 2.2 percent of the trips to change mode of travel. Also, it is significant that 8.9 percent of the automobile drivers made trips for the purpose of serving passengers. All of the serve-passenger trips were made by drivers of automobiles.

In addition to the cases just cited, there are several interesting facts regarding individual cities, which were concealed when trip purposes and modes of travel were grouped. For example, in Columbus, Ga., Baltimore, Md., Charleston, S. C., Reading, Pa., and Grand Rapids, Mich., over 10 percent of the automobile-driver trips were for the purpose of transacting business. In Pontiac, Mich., and Sacramento, Calif., 14 percent of the automobile-driver trips were to serve passengers. The fact that 11 percent of the automobile-driver trips and 9 percent of the mass-transit trips in San Juan, P. R., were for the purpose of eating is due largely to the prevalent local custom of returning home for lunch at midday.

In the category of trips for the purpose of changing mode of travel, several unusual situations occurred in individual urban areas. These trips accounted for 10 and 25 percent of the total streetcar- and bus-passenger trips in Norristown and Philadelphia, Pa., respectively. Also, in Philadelphia, change-mode trips amounted to 60 percent of the subway- or elevated-railway passenger trips and 24 percent of the train-passenger trips.

Over 12 percent of the streetcar- and bus-passenger trips were to school in Madison,

Table 18.—Correlation coefficients computed for certain types of trips and related household characteristics in 49 cities [1]

Mode of travel or purpose of trip	Household characteristic	Correlation coefficient
Mode of travel:		
All modes	Dwelling units	0.987
Automobile driver	Automobiles owned	.975
Mass-transit passenger	Persons 5 years of age and over	.941
Purpose of trip:		
Work and business	Dwelling units	.989
Social and recreation	Automobiles owned	.968
Shop	do	.979
Miscellaneous	Persons 5 years of age and over	.916
Home	Dwelling units	.985

[1] Scatter diagrams, except for social-recreational, miscellaneous, and home trip purposes, are presented in figures 12–16.

Wis., Pontiac, Mich., Sacramento, Calif., and in Phoenix and Tucson, Ariz. Trips to transact business accounted for 11 percent of all taxi-passenger trips in Charleston, S. C., and in Salt Lake City, Utah, and 13 percent in Seattle, Wash.

With regard to modes of travel, again there are individual city exceptions, which were absorbed in the grouping procedure. Among the more important variations which should be mentioned is the case of Washington, D. C., where taxi passengers accounted for almost 3 percent of all trips. Also truck and taxi passengers combined accounted for over 3 percent of the total trips in Baltimore, Md., and Macon, Ga. Finally, train-passenger trips amounted to 5 percent of the total trips in Newark, N. J., and 2 percent in Philadelphia.

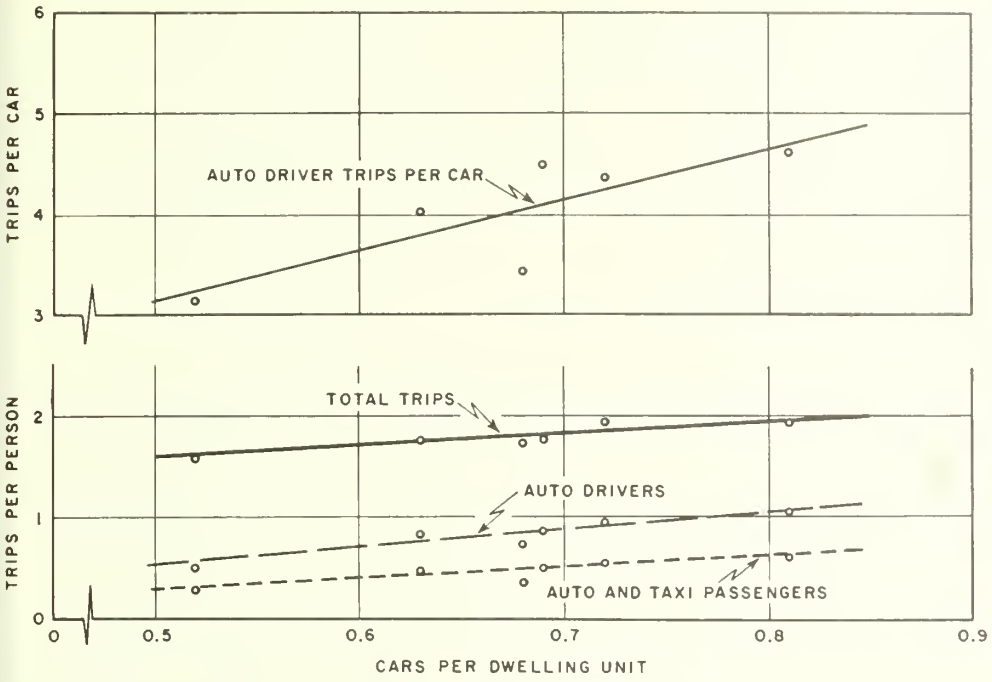

Figure 11.—Trips per person and trips per automobile related to automobiles owned per dwelling unit.

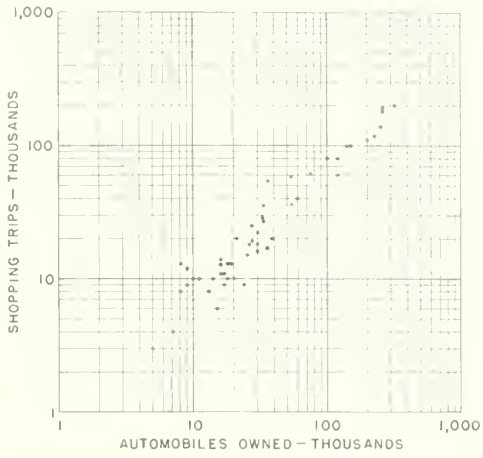

Figure 16.—Number of shopping trips related to number of automobiles owned.

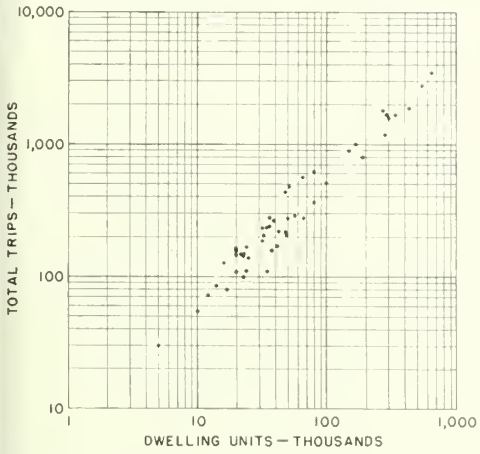

Figure 12.—Number of trips related to number of dwelling units.

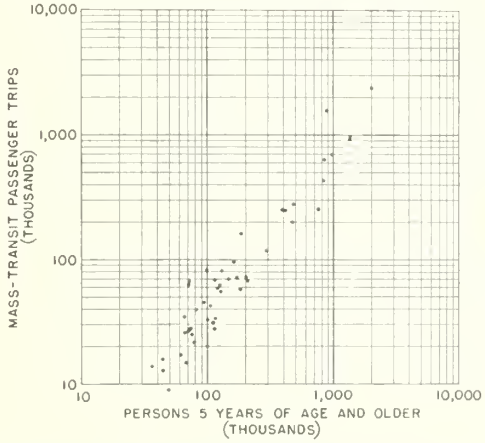

Figure 14.—Number of mass-transit passenger trips related to number of persons 5 years of age and over.

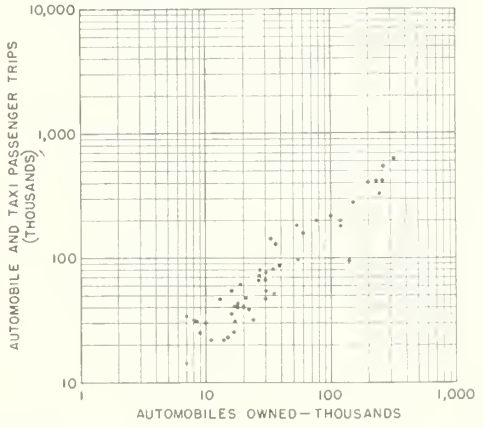

Figure 17.—Number of automobile- and taxi-passenger trips related to number of automobiles owned.

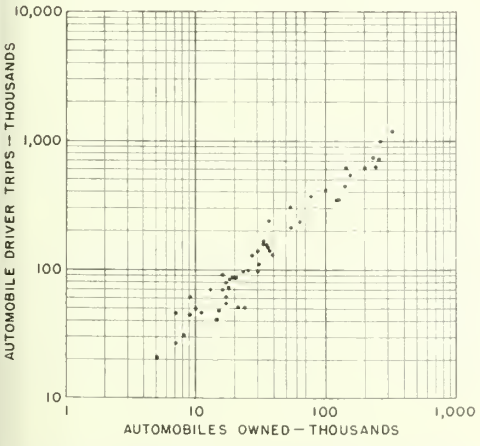

Figure 13.—Number of automobile-driver trips related to number of automobiles owned.

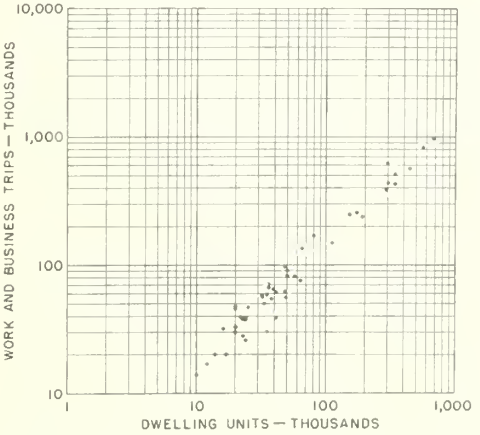

Figure 15.—Number of work and business trips related to number of dwelling units.

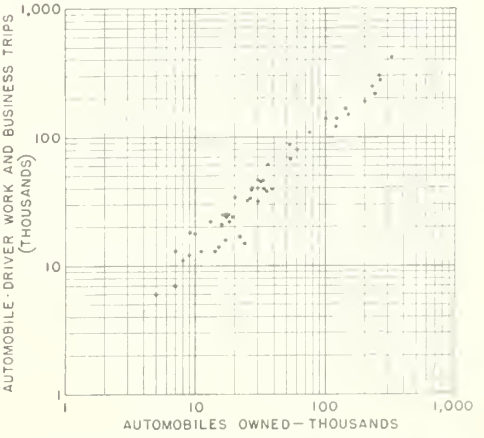

Figure 18.—Number of automobile-driver work and business trips related to number of automobiles owned.

Observations Concerning Urban Traffic Volume Patterns in Tennessee

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported [1] by BORIS B. PETROFF, Head, Traffic Inventory Section, and ANTHONY P. KANCLER, Statistician

The accuracy of estimates of traffic volumes on rural roads in 28 States has been determined in previous studies by probability measures. The production characteristics of the various traffic-counting methods were evaluated, and in the majority of these States efficiency was improved by appropriate changes in procedures. An important conclusion drawn from the rural studies was that traffic-volume sampling variations were relatively small and could be effectively controlled.

Intuitively it has been recognized that in some aspects of traffic volume patterns on city streets a greater uniformity exists than on rural roads, and traffic volumes can be effectively sampled and accurately interpreted by even simpler statistical control devices. It was not, however, until the State of Tennessee had undertaken comprehensive urban traffic volume research that facts began to replace opinions.

The findings of the present study support the judgment previously exercised in Tennessee in the use of some of the procedures and indicate the possibility of improving others. But in addition to the local benefits, the Tennessee studies provide an invaluable background upon which other States and cities can develop efficient urban traffic-counting procedures.

SINCE 1954, 55 continuous-count traffic recorders at 52 locations have been installed in Tennessee cities in order to study the characteristics of urban traffic volumes. The recommendations of the Highway Research Board Committee on Urban Volume Characteristics were used as guides for the selection of locations for these recorders. The 1956 data at 30 locations in 13 cities were analyzed in studies for machine counts, and 1955 data at 33 locations were used in the analysis for manual counts.

To determine the actual annual average daily number of vehicles, hereinafter called ADT, at a particular point on a road or street would require continuous counting for 365 days. On the other extreme, a qualified person could make an estimate without counting, just from general knowledge of the situation. The latter method usually would not be considered acceptable because of the suspected lack of accuracy. Since an exact determination is seldom possible, it becomes axiomatic that the ADT estimates are based on sampling, and the cost of obtaining these estimates must be related to their accuracy. The problem therefore is to find means of measuring the accuracy of ADT estimates obtained by various methods of sampling traffic volumes. The measures employed in the Tennessee studies made use of the configuration of similar patterns of repetition in the mass movement of people and the concepts of probability of these repetitions.

At the present time, only a few basic analyses have been undertaken to aid in the evaluation of existing sampling procedures and to provide essential measures in the development of new traffic-counting schedules. The present as well as other possible schedules were presumed to be based on the assumption that a sample weekday count is representative of the average weekday volume of traffic during the month of the sample count. Therefore, this basic assumption was evaluated and the size of the standard error was estimated. The standard error is a measure of the dispersion about their averages of all possible estimates which are based on samples of a given size. Although the mathematics of probability do not require the knowledge of the true values in these studies, the true (or practically true) values are available at the continuous-count recorders and are therefore used as the basis for measuring errors of estimates developed by sampling.

Conclusions

The following conclusions were reached regarding the observations of 30 urban traffic-counting stations in 13 Tennessee cities:

1. Traffic counts of 24-hour duration on weekdays may be assumed to represent the annual average daily traffic volumes with certain limitations, some of which are subsequently referred to in the discussion of the St. Louis and Detroit studies. Although previous studies have indicated that this assumption may result in an overestimate, the error is within practical limits of acceptance.

2. The monthly variations of traffic are very uniform for the 30 continuous-count stations. The predominant majority of the ratios of ADT to the daily averages at individual stations fall within the ± 10-percent range from the respective monthly means. The standard deviation for the Tennessee urban stations was ± 5.2 percent. Earlier studies in St. Louis and Detroit showed standard deviations of approximately ± 6.0 percent. It appears that confidence limits could be set so that a range lower than ± 10 percent could be achieved if populations could be identified in urban areas. Heterogeneous populations can be separated on the basis of parameters showing similar configurative patterns or selected maximum ranges of deviation.

3. The goodness-of-fit tests as applied to the Gaussian or normal curve can be used to detect heterogeneous populations. These tests include the chi-square and Fisher's g_1 and g_2 statistics. Samples may be taken from heterogeneous populations, and with proper statistical safeguards that samples are representative of the original population they will give satisfactory results. The statistical safeguards are the F- and T-tests.

4. It can be stated from the studies that the 30-station mean monthly adjustment factors could be satisfactorily used. Furthermore, practically the same factors could be obtained from the data for 6 or 7 stations randomly selected. The tests indicate the possibility of refinements in the accuracy of adjustments for monthly variations. Such refinements would require identification of populations which is a costly operation. Even if this were accomplished, the study of Nashville which is subsequently described would indicate that the improvement in the accuracy of estimates of ADT, when based on 24-hour weekday samples, could hardly be expected to reduce the value of the standard deviation by more than 1 percent.

5. Satisfactory estimates of 24-hour weekday traffic volumes can be obtained from weekday counts of 4-hour duration which include either the morning or afternoon hour of peak traffic volume.

6. Differences in social and economic characteristics, upon which the selection of the locations of continuous-count recorders was

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

based, did not seem to influence to any great extent the monthly variations of traffic volumes. However these characteristics should not be disregarded in future studies as they may be found to be significant in other measures of traffic.

Selection of Traffic Stations

In the selection of locations for the 55 continuous-count traffic recorders, State officials have followed in general the recommendations developed by the Committee on Urban Volume Characteristics of the Highway Research Board. These committee suggestions as interpreted by code for Tennessee are as follows:

A. Distribution by city characteristics:
  I. By dominant economic base (as described on pages 37 and 48 of the 1950 Municipal Yearbook):
    (a) Manufacturing and industrial, including diversified manufacturing, mining, and transportation.
    (b) Retail, including diversified retail.
    (c) Wholesale.
    (d) Resort.
    (e) Education.
    (f) Government.
    (g) Dormitory.
  II. By population size (1950 census):
    (a) 1,000,000 and over
    (b) 500,000–1,000,000
    (c) 250,000–500,000
    (d) 100,000–250,000
    (e) 50,000–100,000
    (f) 25,000–50,000
    (g) 10,000–25,000
    (h) Under 10,000

B. Location by street classification:
  I. By traffic function:
    (a) Major or arterial streets:
      1. Radials that are part of primary State highways.
      2. Radials that are not part of primary State highways.
      3. Crosstown (or rings) connecting two or more major radials.
    (b) Secondary streets:
      1. Radials and crosstowns.
      2. Local, commercial, and industrial.
      3. Local and residential.
  II. By average overall speed range in peak period:
    (a) 5–15 miles per hour.
    (b) 15–25 miles per hour.
    (c) 25–35 miles per hour.
    (d) 35–45 miles per hour.

The coding of urban continuous-count stations according to these classifications is shown in table 1.

Data for one complete year of operations, 1956, were available for 30 locations scattered throughout 13 cities. Table 2 shows the distribution of these stations by cities. It is noted in table 1 that these cities vary in population from 514 in Decaturville to over 400,000 in Memphis.

For the purpose of statistical analysis three tabulating cards were developed: Nos. 21 and 31 as shown in figure 1, and the general card, the code sheet of which is shown as figure 2.

Table 1.—Tennessee cities in which continuous-count traffic stations were located

City	Population	City characteristics [1]	Station No.	City street classification [2]
Nashville	176,170	A–I (a) (b) (c) (e) (f), A–II (d)	500	B–I (a) 1, B–II (c).
			501	B–I (b) 3, B–II (b).
			502	B–I (a) 1, B–II (c).
			503	B–I (b) 1, B–II (c).
			504	B–I (a) 2, B–II (b).
			505	B–I (a) 2, B–II (c).
Memphis	407,439	A–I (a) (b) (c) (e), A–II (c)	506	B–I (a) 1, B–II (c).
			507	B–I (b) 2, B–II (a).
			508	B–I (b) 1, B–II (b).
			509	B–I (a) 2, B–II (c).
			510	B–I (b) 3, B–II (b).
			511	B–I (a) 3, B–II (c).
Knoxville	124,769	A–I (a) (b) (c) (e), A–II (d)	512	B–I (b) 1, B–II (b).
			513	B–I (b) 2, B–II (a).
			514	B–I (a) 1, B–II (c).
			515	B–I (a) 1, B–II (c).
			516	B–I (b) 3, B–II (b).
			551	B–I (a) 1, B–II (d).
Johnson City	28,337	A–I (b) (c) (e), A–II (f)	517	B–I (a) 2, B–II (c).
			518	B–I (b) 3, B–II (b).
Morristown	13,151	A–I (a) (b) (c), A–II (g)	519	B–I (b) 2, B–II (a).
			520	B–I (a) 1, B–II (c).
Crossville	2,291	A–I (b) (c), A–II (h)	521	B–I (a) 1, B–II (c).
Rockwood	4,272	A–I (a) (b) (c), A–II (h)	522	B–I (b) 1, B–II (a).
McMinnville	7,577	A–I (a) (b) (c), A–II (h)	523	B–I (b) 1, B–II (a).
Columbia	10,911	A–I (a) (b) (c), A–II (g)	524	B–I (b) 1, B–II (b).
			525	B–I (a) 1, B–II (c).
Jackson	33,354	A–I (a) (b) (c) (e), A–II (f)	526	B–I (a) 1, B–II (c).
			527	B–I (a) 3, B–II (b).
Dyersburg	12,063	A–I (b) (c), A–II (g)	528	B–I (b) 1, B–II (a).
			529	B–I (b) 1, B–II (b).
Dresden	1,509	A–I (b), A–II (h)	530	B–I (a) 1, B–II (c).
Waverly	2,410	A–I (b), A–II (h)	531	B–I (b) 3, B–II (b).
Decaturville	514	A–I (b), A–II (h)	532	B–I (b) 1, B–II (b).
Rogersville	2,670	A–I (b) (c), A–II (h)	533	B–I (a) 1, B–II (c).
Kingsport	19,609	A–I (a) (b) (c), A–II (g)	534	B–I (b) 3, B–II (b).
			535	B–I (b) 2, B–II (a).
Athens	10,103	A–I (b) (c), A–II (g)	536	B–I (b) 3, B–II (b).
			537	B–I (b) 1, B–II (b).
Chattanooga	131,041	A–I (a) (b) (c), A–II (d)	538	B–I (a) 2, B–II (c).
			539	B–I (b) 3, B–II (b).
			540	B–I (a) 3, B–II (b).
			541	B–I (a) 1, B–II (c).
			542	B–I (b) 2, B–II (b).
Bolivar	2,429	A–I (b) (c), A–II (h)	543	B–I (a) 1, B–II (c).
Humboldt	7,426	A–I (b) (c), A–II (h)	544	B–I (a) 1, B–II (c).
			545	B–I (a) 3, B–II (c).
Union City	7,665	A–I (b) (c), A–II (h)	546	B–I (b) 1, B–II (b).
			547	B–I (a) 1, B–II (c).
Shelbyville	9,847	A–I (b) (c), A–II (h)	548	B–I (a) 1, B–II (c).
			549	B–I (a) 2, B–II (c).
Lewisburg	5,312	A–I (b) (c), A–II (h)	550	B–I (a) 1, B–II (b).

[1] Economic characteristics and population groups. Explanation of codes is given in the text on the left.
[2] Traffic functions of streets and average speeds. See codes given in the text on the left.

Procedure

The sampling error of 24-hour weekday (Monday through Friday) counts, which were distributed throughout all months of the year, was computed for the six stations in Memphis as shown in table 3. The mean coefficient of variation of ±5.9 percent denotes that when the traffic volume for a 24-hour period on a given weekday was compared with the average 24-hour weekday traffic during that month at that point, then, based on a normal distribution, it could be expected that approximately two-thirds of such 24-hour weekday counts would not differ by more than ±5.9 percent from the respective monthly means, and 95 percent of such counts should not differ from their respective monthly means by more than twice the value of the coefficient of variation, or ±11.8 percent. It is noteworthy that similar tests conducted by the Bureau of Public Roads during 1957 on the 1954 dat for 12 stations in St. Louis, Mo., resulted in standard deviation of ±5.4 percent; an studies made on 1954–55 data (April through November) at 10 stations in Detroit, Mich. indicated a standard deviation of ±6. percent.

If the truest adjustment ratio of ADT to the average weekday of the month (the ratio derived from the same station from which the sample was taken) were applied to the sample to estimate the ADT, the measure of error in such estimates would still be expressed by the coefficient of variation of ±5.9 percent. Since the mean ratio value of the various tests based on ADT is unity (1.00), the coefficient of variation is equal to the standard deviation. The significance of the measure of standard deviation in these cases is practically synonymous with that of the coefficient of variation. Thus, the ±5.9-percent measure of the

Table 2.—Ratios of annual average daily traffic to average daily traffic volumes for 30 stations in 13 Tennessee cities during 1956

City	Station number	Ratios of annual average daily traffic to average daily traffic volumes and deviations [1] of each ratio from the mean monthly ratio																					
		January		February		March		April		May		June		July		August		September		October		November		December	
		Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation	Ratio	Deviation
Nashville	500	1.10	−1	1.02	−4	1.01	−1	0.97	−3	0.97	0	0.99	3	0.96	−1	0.98	2	1.02	4	1.01	2	1.02	2	0.97	−2
	501	1.11	0	.99	−7	.99	−3	1.01	1	.96	−1	1.05	9	1.02	5	1.02	6	1.01	3	.97	−2	.97	−3	.93	−6
	502	[2]		[2]		1.01	−1	.95	−5	.97	0	.94	−2	.88	−9	.87	−9	1.00	2	1.03	4	1.05	5	.99	0
	503	1.22	11	1.01	0	.99	−3	.95	−5	.95	−2	.94	−2	.98	1	.98	2	.98	0	.99	0	1.01	1	1.00	1
	504	1.07	−4	1.04	−2	1.03	1	1.00	0	.99	2	1.01	5	1.03	6	.95	−1	.94	−4	.95	−4	1.01	1	1.00	1
	505	1.09	−2	1.04	−2	1.03	1	1.03	3	.97	0	.93	−3	.94	−3	.94	−2	.99	1	1.05	6	1.03	3	1.00	1
Memphis	506	1.09	−2	1.13	7	1.06	4	1.05	5	.95	−2	.91	−5	.90	−7	.87	−9	.87	−11	.87	−12	.91	−9	1.02	3
	507	1.00	−11	1.06	0	1.05	3	1.02	2	1.00	3	.92	−4	.93	−4	.98	2	1.00	2	1.01	2	1.01	1	1.04	5
	508	1.05	−6	1.07	1	1.02	0	.97	−3	.95	−2	.97	1	.99	2	.98	2	[2]		.97	−2	.98	−2	.93	−6
	509	1.02	−9	.96	−10	.98	−4	.94	−6	.93	−4	.94	−2	1.08	11	1.05	9	1.05	7	1.04	5	1.05	5	[2]	
	510	1.08	−3	1.03	−3	1.00	−2	1.03	3	1.02	5	.99	3	1.01	4	.96	3	.96	−2	.97	−2	.97	−3	.95	−4
	511	1.19	8	1.06	0	1.05	3	1.11	11	.96	−1	.92	−4	.95	−2	.89	−7	.89	−9	.99	0	.95	−5	1.13	14
Knoxville	512	1.07	−4	1.05	−1	1.03	1	1.01	1	1.03	6	1.05	9	.86	−11	1.00	4	1.02	4	.93	−6	.96	−4	1.04	5
	513	1.24	13	1.15	9	1.11	9	1.04	4	.95	−2	.90	−6	.91	−6	.88	−8	.99	1	.99	0	.96	−4	1.00	1
	515	1.11	0	1.04	−2	.96	−6	.98	−2	.98	1	.94	−2	.93	−4	.94	−2	1.02	4	1.03	4	1.08	8	1.03	4
Johnson City	517	1.15	4	1.05	−1	1.02	0	1.00	0	.96	−1	.98	2	.96	−1	.94	−2	.96	−2	.98	−1	1.02	2	1.00	1
	518	1.28	17	1.11	5	1.05	3	.98	−2	.95	−2	.95	−1	.93	−4	.80	−16	.86	−12	.99	0	[2]		1.05	6
Morristown	519	1.09	−2	1.08	2	1.03	1	1.01	1	.98	1	.95	−1	.95	−2	.94	−2	.97	−1	1.00	1	1.01	1	1.03	4
	520	1.13	2	1.14	8	1.10	8	1.03	3	1.00	3	.97	1	1.06	9	.99	3	1.01	3	.89	−10	.85	−15	.93	−6
Crossville	521	1.21	10	1.16	10	1.08	6	1.05	5	.96	−1	.86	−10	.88	−9	.84	−12	.91	−7	1.01	2	1.11	11	1.11	12
Rockwood	522	1.03	−8	1.02	−4	1.00	−2	1.00	0	.99	2	.94	−2	1.00	3	.98	2	.99	1	1.03	4	1.05	5	.98	−1
McMinnville	523	1.13	2	1.10	4	1.03	1	1.03	3	1.00	3	1.03	7	.97	0	.97	1	.99	1	.96	−3	.94	−6	.91	−8
Columbia	524	1.08	−3	1.01	−5	.97	−5	1.00	0	.96	−1	1.04	8	1.07	10	1.04	8	.94	−4	.97	−2	.97	−3	.98	−1
	525	1.12	1	1.06	0	1.00	−2	1.00	0	.96	−1	.99	3	.99	2	.97	1	.95	−3	.98	−1	.99	−1	1.01	2
Jackson	526	1.03	−8	.99	−7	.95	−7	.97	−3	1.01	4	1.01	5	1.00	3	[2]		.99	1	.99	0	1.00	0	.96	−3
	527	1.05	−6	1.03	−3	1.01	−1	.98	−2	.97	0	.99	3	1.04	7	1.03	7	.99	1	.97	−2	.98	−2	.97	−2
Dyersburg	528	1.09	−2	1.02	−4	.97	−5	1.00	0	.97	0	.94	−2	.94	−3	.96	0	1.00	2	1.03	4	1.07	7	1.03	4
	529	1.07	−4	.97	−9	.93	−9	.96	−4	.88	−9	.89	−7	1.06	9	1.05	9	1.00	2	1.06	7	1.11	11	1.11	12
Dresden	530	1.15	4	1.10	4	.93	−9	.93	−7	.99	2	.98	2	.98	1	1.00	4	1.02	4	1.09	10	1.05	5	.86	−13
Decaturville	532	1.17	6	1.10	4	1.14	12	1.12	12	.95	−2	.97	1	.94	−3	.90	−6	1.04	6	1.00	1	.94	−6	.86	−13
Mean monthly ratio		1.11		1.06		1.02		1.00		.97		.96		.97		.96		.98		.99		1.00		.99	
Σd (net)			3		10		−7		12		1		9		4		−11		−6		5		5		11
Σd^2 (total)			1,305		756		719		564		245		649		990		1,111		634		615		947		1,181

[1] Deviation $(d)=(X_1-X)100$, where X_1 is the ratio of the station's annual ADT to the average daily traffic of the month and X is the mean monthly ratio for all stations.
[2] Values were unacceptable for various reasons.

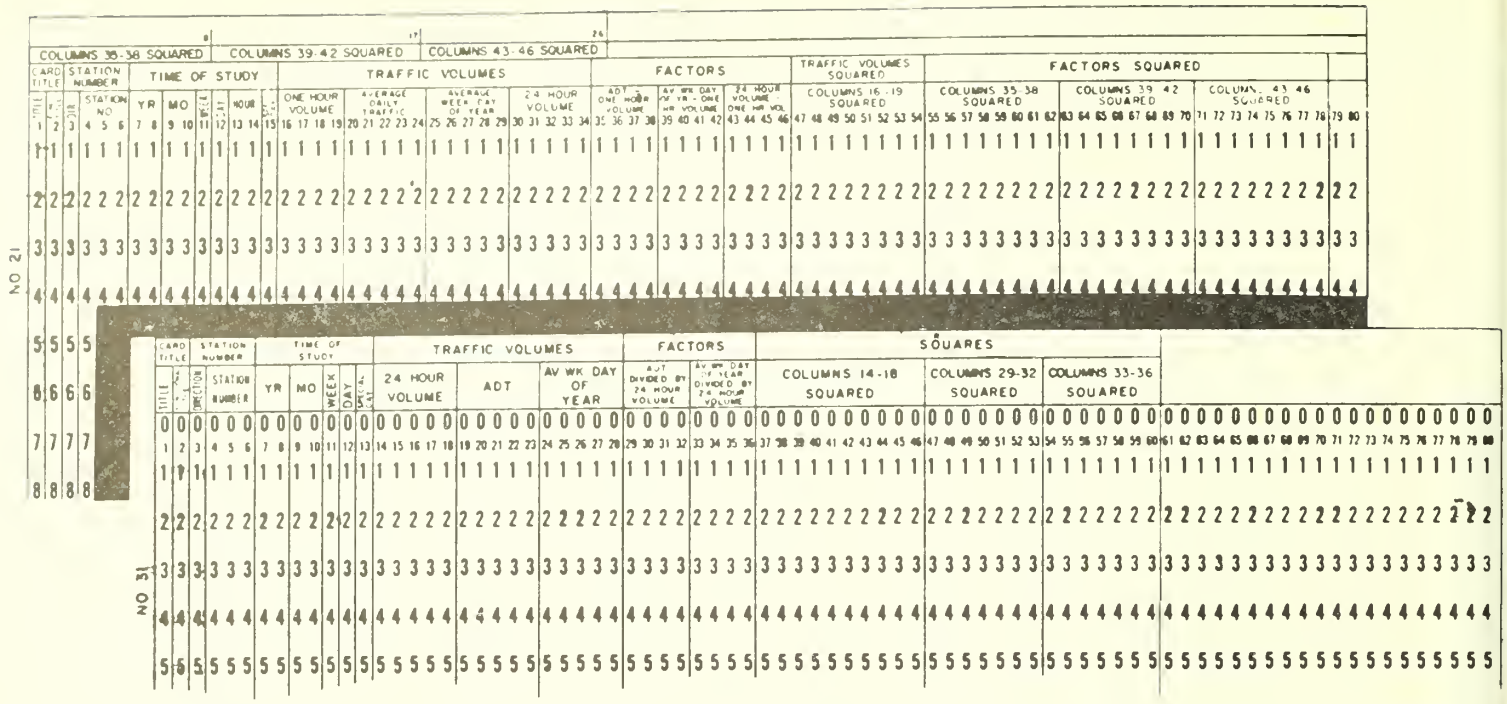

Figure 1.—Tabulating cards used by the Tennessee Department of Highways and Public Works for summarizing urban traffic data.

sampling error is the minimum that can be expected in the distribution of errors in ADT estimates in this particular study; that is, when these estimates are based on adjustment ratios computed in terms of ADT to the average weekday of the month from any other source, it generally can be expected that the measure of the error in ADT estimates will be greater than the measure of the sampling error.

	Card Column Number	Sunday	Monday	Tuesday	Wednesday	Thursday	Friday	Saturday
Station Number	1 - 2 - 3 - 4							
Day Of Week	5	7	1	2	3	4	5	6
Month Of Count	6 - 7							
Day Of Month	8 - 9							
Year	10 - 11							
Traffic Volume 6 A.M. - 7 A.M.	12 - 15							
7 - 8	16 - 19							
8 - 9	20 - 23							
9 - 10	24 - 27							
10 - 11	28 - 31							
11 - 12 Noon	32 - 35							
12 Noon - 1 P.M.	36 - 39							
1 - 2	40 - 43							
2 - 3	44 - 47							
3 - 4	48 - 51							
4 - 5	52 - 55							
5 - 6	56 - 59							
6 - 7	60 - 63							
7 - 8	64 - 67							
8 - 9	68 - 71							
Total 24 Hour Volume	72 - 76							
Peak Hour Volume	77 - 80							

Figure 2.—Coding sheet for automatic traffic recorder data.

The 24-hour weekday counts were adjusted to the ADT by application of appropriate factors. These factors were obtained from a group of stations having similar patterns of monthly variations of traffic volumes. They should be in terms of ratios of ADT to the weekday traffic of the respective months. Since the factors were based on group values, the resulting group mean values are characterized by differences between the individual station data and the group mean data. Thus, factors were another source of error contributing to the error in the ADT estimates. The material readily available did not permit the evaluation of the error in such factors. However, a reasonable approximation was available in terms of the ratios of ADT to the average daily volume for each month for the 30 stations in 13 cities. These ratios permitted the measurement of monthly variations and the comparisons of these variations among stations. The ratios and comparisons are shown in table 2. It is noted that the overwhelming majority of the monthly ratios vary from the respective means of the 30 stations by ± 10 percent or less, and the standard deviation of these differences is ± 5.2 percent.

$$S = \sqrt{\frac{1}{N-1}\left[\Sigma d^2 - \frac{1}{N}(\Sigma d)^2\right]} =$$

$$\sqrt{\frac{1}{354-1}\left[9{,}716 - \frac{1}{354}(16)^2\right]} = \sqrt{27.52} = \pm 5.2$$

By comparison with the spread of seasonal variation usually encountered on rural roads, the extremely narrow range observed in this study and the implications of these observations as regards traffic survey costs were given special attention in the analysis.

Table 3.—Errors in sampling of Memphis traffic volumes for 24-hour periods on weekdays in 1955 [1]

Station number and item	January	February	March	April	May	June	July	August	September	October	November	December
Station No. 506:												
Number of weekdays	12	19	18	24	21	20	20	17	19	22	21	22
Average weekday volume	17,668	18,703	20,402	21,684	22,061	23,836	24,378	24,174	23,712	24,115	22,297	22,831
Coefficient of variation	10.8	3.3	3.9	3.9	3.1	2.4	2.8	3.2	8.4	2.7	3.5	8.8
Station No. 507:												
Number of weekdays	17	20	16	10	20	8	24	20	19	22	19	24
Average weekday volume	9,172	9,049	9,660	10,203	11,429	10,695	10,608	10,221	10,606	12,331	11,714	11,475
Coefficient of variation	4.8	4.5	3.8	5.3	12.6	3.9	4.6	4.7	4.4	7.9	4.2	6.2
Station No. 508:												
Number of weekdays	10	17	16	22	20	11	24	15	19	25	19	24
Average weekday volume	15,014	11,777	12,355	12,478	12,607	13,009	12,694	12,643	13,339	12,506	13,198	13,040
Coefficient of variation	20.7	3.8	3.6	5.6	7.2	4.3	3.9	3.6	7.1	11.2	5.6	6.4
Station No. 509:												
Number of weekdays	16	20	16	20	14	16	24	17	15	23	16	20
Average weekday volume	18,095	17,984	18,222	18,188	18,772	19,366	18,741	18,585	19,475	22,731	23,292	23,550
Coefficient of variation	2.2	4.7	3.6	6.7	10.8	3.5	3.7	4.2	14.4	1.7	4.3	8.0
Station No. 510:												
Number of weekdays	19	20	14	23	20	20	24	19	12	23	19	24
Average weekday volume	8,385	7,527	7,903	7,942	7,559	7,383	8,173	7,598	7,962	7,242	7,232	7,280
Coefficient of variation	1.8	5.1	4.9	4.2	4.9	3.8	10.6	5.9	4.4	7.2	2.6	7.3
Station No. 511:												
Number of weekdays	15	18	18	23	13	12	18	20	16	14	13	24
Average weekday volume	29,575	24,549	25,333	27,038	25,106	27,775	27,375	25,410	25,578	28,721	26,624	25,940
Coefficient of variation	8.9	4.6	4.2	4.0	8.5	7.9	9.8	5.9	11.7	5.6	10.0	7.9

[1] Mean coefficient of variation = 426.2/72 = ±5.9 percent.

Monthly Expansion Factors

Experience with rural traffic counts [2] indicates that when monthly factors fall within the ±10-percent range of the group mean, then the effect of added amount of error to the sampling error of the 24-hour sample in the estimates of ADT is very small. Thus, it appeared that single monthly expansion factors, which are the means of the 30 stations, could be used in Tennessee for the expansion of 24-hour weekday sample counts so that the resulting errors in ADT estimates would not be much larger than those which are expressed by the standard deviation of ±5.9 percent.

[2] *Experience in application of statistical method to traffic counting*, by Boris B. Petroff. PUBLIC ROADS, vol. 29, No. 5, Dec. 1956.

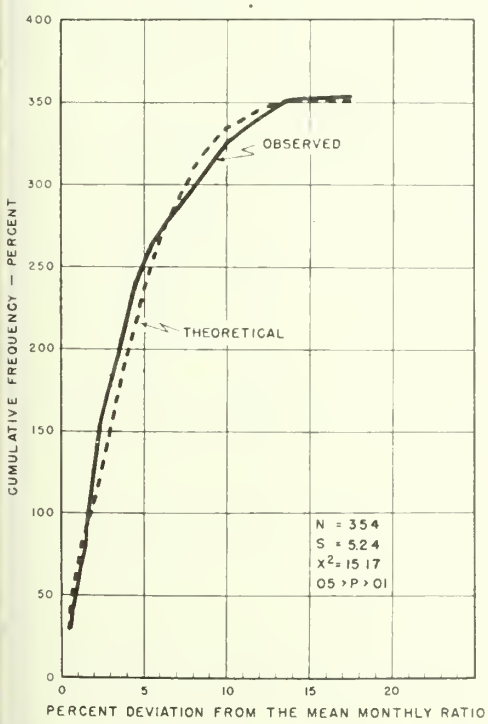

Figure 3.—Comparison of observed and theoretical distributions of deviations of individual ratios of ADT from the mean monthly ratios.

The chi-square test of these data (standard deviation ±5.2 percent) showed a probability level between 5 and 1 percent. Considering "good fit" within the range from 5 to 95 percent, the goodness of fit was not quite acceptable. The normal distribution is applicable only when chance forces are in operation. In this instance the normal distribution of the observed values is borderline, which indicates the possibility of forces or heterogeneous populations causing results not due to chance alone

The computation of chi square given in table 4 and the values obtained are presented in figure 3. The tendency for the traffic observations to concentrate bimodally on either side of the mean contributes to the low chi-square probability level.

Three random samples of 6, 5, and 4 stations were taken from the data for 30 stations; the respective standard deviations (S) were ±5.93, ±4.70, and ±2.53. The F-test related the variance (S^2) of each of the three random samples to the variance of the 30-station data and expressed the probability level of the relation. The test showed that the 5- and 6-station random samples yielded stable results, whereas variations for the 4-station random sample were so much greater as to be unreliable. The formula for the F-test is as follows:

$$F=\frac{S_1^2}{S_2^2}$$

Where:

S_1^2 = The larger variance.

S_2^2 = The smaller variance.

Another test for conformity, the T-test, related the significance of the differences in the monthly means of each of the three random samples to the monthly means of the 30-station data, but here the differences were not significant for all three.

$$T=\frac{\overline{X}_1-\overline{X}_2}{S_{(\overline{X}_1-\overline{X}_2)}}$$

Where:

$$S_{(\overline{X}_1-\overline{X}_2)}=\sqrt{\frac{(N_1+N_2)(\Sigma d_1^2+\Sigma d_2^2)}{N_1N_2[(N_1-1)+(N_2-1)]}}$$

$\overline{X}_1$ = Monthly mean of sample having N_1 observations per month.

$\overline{X}_2$ = Monthly mean of sample having N_2 observations per month.

Σd_1^2 = Sum of the squares of the deviations of N_1 observations from the monthly mean.

Σd_2^2 = Sum of the squares of the deviations of N_2 observations from the monthly mean.

A chi-square test on each of the three random samples conformed with normal curve requirements. Random samples are not always representative since they are subject to the laws of chance. In this particular instance, the use of the 4-station random sample would appear to be the least satisfactory.

Table 4.—Chi-square test of deviations of weekday ratios of ADT from the monthly averages

Class interval	X	$\frac{X}{S}$	Cumulative frequency theoretical	Theoretical f_t	Cumulative frequency observed	Observed f_o	f_o-f_t	$(f_o-f_t)^2$	$\frac{(f_o-f_t)^2}{f_t}$
0.00–1.99	2.00	0.38	104.8	104.8	88	88	−16.8	282.24	2.69
2.00–3.99	4.00	.76	195.7	90.9	197	109	18.1	327.61	3.60
4.00–5.99	6.00	1.15	265.4	69.7	260	63	−6.7	44.89	.64
6.00–7.99	8.00	1.53	309.4	44.0	296	36	−8.0	64.00	1.45
8.00–9.99	10.00	1.91	334.1	24.7	325	29	4.3	18.49	.75
10.00–11.99	12.00	2.29	346.2	12.1	340	15	2.9	8.41	.70
12.00–13.99	14.00	2.67	351.3	5.1	350	10	4.9	24.01	4.71
14.00–15.99	16.00	3.05	353.2	1.9	352	2			
16.00–17.99	18.00	3.44	353.8	.6	354	2	1.3	1.69	.63
18.00 and over	-----	----	354.0	.2	354	0			

Degrees of freedom = 8−2=6, 0.05 > P > 0.01, S = ±5.24 χ^2 = 15.17

Grouping Stations

It was previously mentioned that the Tennessee 30-station data could have had a heterogeneous population. The following method was used to divide the original data into population groups having similar characteristics or, as in this case, pattern conformity: (1) An array of the 30 stations based on their ratio values was set up for each month of the year, as shown in table 5; (2) the median and quartile values for each month were determined; (3) arbitrary values were assigned to the quartile position of each station for each month, as shown in table 6, thereby setting up a configurative pattern for each station's relationship to all other stations; and (4) stations were grouped into five categories according to individual patterns.

Group I.—Stations having a relatively small amplitude of deviations from the monthly medians.

Group II.—Stations tending to deviate greatly from the monthly median values for the first 6 months of the year.

Group III.—Stations tending to deviate greatly from the monthly median values for the last 6 months of the year.

Group IV.—Stations having monthly values occurring within the interquartile range for more than 9 months of the year. In a normal distribution the interquartile range is the 50-percent probability level as contrasted with the standard deviation of 68 percent. For the Tennessee data, this range was approximately ±5 percent and the group included 12 of the 30 stations.

Group V.—A station having monthly values closest to the monthly mean or median of all stations was selected. Using the monthly mean or median values of this station (No. 519) as a control, all stations having values falling within ±10 percent of the control values were included in group V. Although this method does not necessarily separate populations from a heterogeneous group, it does eliminate extreme values and trouble spots which probably should have been eliminated originally for one reason or another.

Table 5.—Frequency distribution of traffic stations by values of the ratio of annual average daily traffic to the average daily traffic volumes for each month [1]

Ratio	January	February	March	April	May	June	July	August	September	October	November	December
0.80								1				
.84								1				
.85											1	
.86						1	1		1			2
.87								2	1	1		
.88					1		2	1				
.89						1		1	1	1		
.90						1	1	1				
.91						1	1		1		1	1
.92						2						
.93			2	1	1	1	*3*			1		3
.94				1		*6*	3	*4*	2		2	
.95			1	2	*6*	2	2	1	1	1	1	1
.96		1	1	1	6		2	1	*2*	1	2	*1*
.97		1	2	*3*	5	*3*	*1*	*2*	1	*5*	*3*	2
.98			1	3	2	2	2	5	1	2	2	2
.99		2	*2*		*3*	*4*	2	*2*	*6*	*5*	1	1
1.00	1		3	*6*	3		2	2	4	2	1	*6*
1.01		1	3	3	1	2	*1*		*2*	3	*4*	1
1.02	1	*3*	*2*	1	1		1	1	4		2	1
1.03	2	2	5	*4*	1	1	1	1		*4*	1	*3*
1.04		3		1		1	1	1	1	1		2
1.05	2	*2*	*3*	2		2		2	1	1	*4*	1
1.06		4	1				2			1		
1.07	*3*	1					2				1	
1.08	2	1	1								1	
1.09	*4*									1		
1.10	1	*3*	1									
1.11	2	1	1	1							2	2
1.12	1			1								
1.13	2	1										1
1.14		1	1									
1.15	*2*	1										
1.16		1										
1.17	1											
1.19	1											
1.21	1											
1.22	1											
1.24	1											
1.28	1											
Number of stations	29	29	30	30	30	30	30	29	29	30	29	29

[1] Numbers in italics indicate the first, second, and third quartile points in each month.

Testing by Statistical Method

To test whether groups I through V belong to significantly different populations, they were checked against each other by the use of the *F*- and *T*-tests. The results showed that groups I, II, and III were distinct populations, and groups I and IV were not significantly different, since their selection was based more or less on the frequency of monthly central tendency. Group IV is a mixed population

Table 6.—Quartile position by month for each station in relation to all stations [1]

Station number	Group number	January	February	March	April	May	June	July	August	September	October	November	December
500	III, IV, V	3	Q_1	2	Q_1	Q_2	Q_3	2	3	4	3	3	2
501	I, V	3	1	Q_1	2	2	4	4	4	Q_2	Q_1	Q_1	1
502	III, IV, V			2	1	Q_2	Q_1	1	1	3	Q_3	Q_3	2
503	I, IV	4	3	Q_1	1	Q_1	Q_1	3	3	2	Q_2	Q_2	Q_2
504	II, V	Q_1	2	3	Q_2	Q_1	4	4	2	1	1	Q_2	Q_2
505	I, IV, V	Q_2	2	3	Q_3	Q_2	1	2	Q_1	Q_2	4	3	Q_2
506	II	Q_2	4	4	4	Q_1	1	1	1	1	1	1	3
507	I, V	1	3	Q_3	3	4	1	Q_1	3	3	3	Q_2	4
508	I, IV, V	1	3	Q_2	Q_1	Q_1	3	3	3		Q_1	2	1
509	III	1	1	1	1	1	Q_3	4	4	4	4	Q_3	
510	II, IV, V	2	2	2	Q_3	4	Q_3	Q_3	Q_2	Q_1	Q_2	Q_2	1
511	II, V	4	3	Q_2	4	2	1	2	1	1	Q_2	1	4
512	I, V	Q_1	Q_2	3	3	4	4	1	4	4	1	1	4
513	II	4	4	4	4	Q_1	1	1	1	Q_2	Q_2	1	Q_2
515	I, IV, V	3	2	1	2	3	Q_1	Q_1	Q_1	Q_1	Q_3	4	Q_2
517	I, IV, V	Q_2	Q_2	Q_2	Q_2	2	3	2	Q_1	4	2	3	Q_2
518	II	4	4	Q_3	2	Q_1	2	Q_1	1	1	Q_2		4
519	I, IV, V	2	3	3	3	3	2	2	Q_1	2	3	Q_2	Q_3
520	II	3	4	4	Q_3	4	3	4	Q_3	Q_3	1	1	1
521	II	4	4	4	4	2	1	1	1	1	3	4	4
522	III, IV, V	1	Q_1	2	Q_2	Q_3	Q_1	3	3	Q_2	Q_3	Q_2	2
523	II	3	Q_3	3	Q_3	4	4	Q_2	Q_2	Q_2	1	1	1
524	I	2	1	1	Q_2	2	4	4	4	1	Q_1	Q_1	2
525	I, IV, V	3	3	2	Q_2	2	Q_3	3	Q_2	1	2	2	3
526	I, V	1	1	1	Q_1	4	4	3		Q_2	Q_2	2	Q_1
527	I, V	1	2	2	2	Q_2	Q_3	4	4	Q_2	Q_1	2	2
528	III, IV, V	2	Q_1	1	Q_2	Q_2	Q_1	2	2	3	Q_3	4	Q_3
529	III	Q_1	1	1	1	1	1	4	4	3	4	4	4
530	III	Q_1	Q_1	1	1	Q_3	3	3	4	4	4	Q_3	1
532	II	4	Q_3	4	4	Q_1	3	2	1	4	3	1	1

[1] Numbers 1-4 shown in the columns for the respective months express the following relation: 1=ratio value$<Q_1$, 2=ratio value$>Q_1<Q_2$, 3=ratio value$>Q_2<Q_3$, and 4=ratio value$>Q_3$

but it can be used when the least variance from the mean is desired. Group V is also a mixed population, tending to resemble groups I and IV; it serves to eliminate undesirable extreme values due to error or forces incompatible with the remainder of the data. The separate populations can be broken down into subpopulations; however, there is danger in accepting the manifestations of a small group of individual stations which may not be truly representative of the whole population group.

The chi-square test was applied to all five groups with satisfactory results for all except groups III and V, indicating that these two groups still had heterogeneous populations and could be divided into more populations.

Another test supplementing the chi-square goodness-of-fit test to the normal curve was also made, namely Fisher's g_1 and g_2 statistics. For each sample, these values are based on the first through the fourth moments of the deviations of the observations from the mean of a frequency distribution where the X-axis is the class interval of the monthly value of the ratio of each station's ADT to the average day of the month, and the Y-axis is the frequency of occurrence. Just as the first and second moments about the mean are measures of the average deviation from the mean and the standard deviation, respectively, so are the first through the third moments used to obtain a measure of asymmetry (g_1) and the first through the fourth moments a measure of the kurtosis, flatness, and/or peakedness (g_2) as compared with the normal curve. The statistics g_1 and g_2 are calculated from the k statistics, which are in turn derived from the sum of the powers, from the second through the fourth, of the deviations from the mean of a frequency distribution. Thus,

$$g_1 = \frac{k_3}{\sqrt{k_2^3}} \text{ and } g_2 = \frac{k_4}{k_2^2}$$

Where:

$$k_2 = S_2/(N-1)$$

$$k_3 = NS_3/(N-1)(N-2)$$

$$k_4 = \frac{N[(N+1)S_4 - 3(N-1)S_2^2/N]}{(N-1)(N-2)(N-3)}$$

S_2 = The sum of the squares of the deviations about the mean.

S_3 = The sum of the cubes of the deviations about the mean.

S_4 = The sum of the fourth powers of the deviations about the mean.

In converting the values of g_1 and g_2 to t values which show the probability levels and significance of the sample in relation to the normal curve, the following formulas were used:

$$t_{g_1} = \frac{g_1}{S^2_{g_1}} \text{ and } t_{g_2} = \frac{g_2}{S^2_{g_2}}$$

Where:

$$S^2_{g_1}(\text{variance of } g_1) = \frac{6N(N-1)}{(N-2)(N+1)(N+3)}$$

$$S^2_{g_2}(\text{variance of } g_2) = \frac{24N(N-1)^2}{(N-3)(N-2)(N+3)(N+5)}$$

N = Number of observations in samples.

An interesting sidelight on the value of g_2 is its use in determining the minimum size of a sample to be taken from a larger sample or population when the value of g_2 of the larger sample is known. The minimum sample size is computed as follows: $\beta_2 = g_2 + 3$, and N (size of sample) $= (\beta_2 - 1)\,41^2$. In the Tennessee 30-station data, the value of g_2 is 0.4477, and assuming the desired coefficient of variation (V) of the standard deviation is equal to 10 percent, $\beta_2 = 0.4477 + 3.000 = 3.4477$. The size of sample $N = 3.4477 - 1/4\ (0.10)^2 = 2.4477/0.04 = 61.2$ months.

Since each station reports for 12 months, the minimum sample required is 61.2 12 or 5 stations. However, this sample of 61.2 months is a random sample distributed over all stations and not clustered in 5 stations. This cluster effect has not yet been investigated, but because of its possible effect the number of stations may have to be raised to 6 or 7.

It has been observed that when the chi-square test for goodness of fit showed weakness, the g_1 and g_2 tests tended to substantiate this weakness. A summary of the results of the various tests for selected groups is shown in table 7.

Table 7.—Summary of various statistical tests for selected station groupings

Comparison	Standard deviation	F-test of variance [1]	T-test of means	Chi-square test [2]	α-criteria probability levels — t_{g_1}	t_{g_2}
All 30 stations	±5.24			0.05 > P > 0.01	P = 0.90	0.10 > P > 0.05
All 30 stations compared with						
6-station random sample		1.28				
5-station random sample		1.24				
4-station random sample		4.29				
Group I stations		2.12				
Group II stations		1.09	(4)			
Group III stations		1.49	(4)			
Group IV stations		3.20				
Group V stations		2.05				
6-station random sample	±5.93			.70 > P > .50	P = .90	.70 > P > .60
6-station sample compared with						
5-station random sample		1.59				
4-station random sample		5.49				
Group I stations		2.71				
Group II stations		1.18				
Group III stations		1.91				
Group IV stations		4.10				
Group V stations		2.70				
5-station random sample	±4.70			.30 > P > .20	.80 > P > .70	.30 > P > .20
5-station sample compared with						
4-station random sample		3.45				
Group I stations		1.70				
Group II stations		1.35				
Group III stations		1.20	(4)			
Group IV stations		2.57				
Group V stations		1.65				
4-station random sample	±2.53			.90 > P > .80	.20 > P > .10	.50 > P > .40
4-station sample compared with						
Group I stations		2.03				
Group II stations		4.66				
Group III stations		2.88	(4)			
Group IV stations		1.34				
Group V stations		2.09				
Group I (13 stations)	±3.60			P = .10	.30 > P > .20	.10 > P > .05
Group I stations compared with						
Group II stations		2.30	(3)			
Group III stations		1.42	(3)			
Group IV stations		1.51				
Group V stations		1.03				
Group II (10 stations)	±5.46			.90 > P > .80	.60 > P > .50	P > .90
Group II stations compared with						
Group III stations		1.62	(3)			
Group IV stations		3.47	(3)			
Group V stations		2.22				
Group III (7 stations)	±4.29			.02 > P > .01	.50 > P > .40	.02 > P > .01
Group III stations compared with						
Group IV stations		2.14	(3)			
Group V stations		1.37	(3)			
Group IV (12 stations)	±2.93			.20 > P > .10	.90 > P > .80	P > .001
Group IV stations compared with						
Group V stations		1.56				
Group V (18 stations)	±3.66			.05 > P > .02	.50 > P > .40	.05 > P > .001

[1] Values exceeding 1.35 are significant at the 5-percent level.
[2] Acceptable probability level is between 0.95 and 0.05.
[3] P values less than 0.05 are considered significantly different from a normal fit.
[4] Difference in means is highly significant at the 5-percent level.

Nashville and Memphis Studies

From the data of 6 stations located in Nashville, 63 random samples of 24-hour duration were selected as shown in table 8. These samples were adjusted to the ADT estimates by application of the 6-station monthly means of ratios of ADT to the respective average weekday traffic volumes as shown in table 9. The differences (errors) of these estimates from their respective true values were expressed by the standard deviation of ±6.7 percent. Recalling that the sampling error of the 24-hour samples was measured by the standard deviation of ±5.9 percent for Memphis, the effect of factorization on the final error is small indeed.

Further, to test the practical meaning of the significance of the observed ±10-percent range of variation in the monthly characteristics of the variations among stations, it was

assumed that no monthly adjustment ratios were available from Nashville stations. Instead, the monthly mean ratios for the 6 stations located in Memphis were used for estimating the ADT in Nashville using the same 63 samples. The standard deviation resulting from this procedure was ±7.2 percent.

The difference between 7.2 and 6.7 percent could hardly be considered of practical significance, and yet it implies the absence of need for Nashville data for the adjustment of samples. At least for this purpose, the six stations in Nashville could be considered unnecessary. Furthermore, the identification of possible different populations as previously discussed could not have had any appreciable practical effect on the accuracy of ADT estimates based on 24-hour weekday samples, as the error could not be expected to fall below the ±5.9 percent standard deviation of sampling.

A comparison of the same 63 sample count with the ADT disclosed that the difference between the sample traffic volumes and the respective ADT volumes were measured by a standard deviation of ±8.7 percent. Considering that the corresponding minimum possible measure was ±5.9 percent, and the best results upon factorization (by Nashville factors) was 6.7 percent, a significant conclusion is derived. If on a 68-percent confidence limit, errors of 9 percent or less would be

Table 8.—Errors in ADT estimates of Nashville traffic for 1956, based on 24-hour weekday samples expanded by mean factors

Month	24-hour weekday volumes	Using mean Nashville factor: Factor	Estimated ADT	Error [1]: Volume	Error [1]: Percent	Using mean Memphis factor: Factor	Estimated ADT	Error [2]: Volume	Error [2]: Percent
Station 500 ADT 26,635									
January	24,821	1.09	27,055	420	1.6	1.06	26,310	−325	−1.2
February	25,528	1.02	26,038	−597	−2.2	1.03	26,294	−341	−1.3
March	25,727	.98	25,212	−1,423	−5.3	1.01	25,984	−651	−2.4
April	27,727	.95	26,341	−294	−1.1	1.01	28,004	1,369	5.1
May	29,876	.92	27,486	851	3.2	.96	28,681	2,046	7.7
June	30,435	.93	28,305	1,670	6.3	.93	28,304	1,669	6.3
July	27,407	.93	25,489	−1,146	−4.3	.96	26,311	−324	−1.2
August		.92				.94			
September		.95				.95			
October	25,680	.95	24,396	−2,239	−8.4	.96	24,653	1,982	−7.4
November	24,853	.97	24,107	−2,529	−9.5	.96	23,859	2,776	−10.4
December		.94				.95			
Station 501—ADT 576									
January	458	1.09	499	−77	−13.3	1.06	485	−91	−15.8
February	594	1.02	606	30	5.2	1.03	612	36	6.3
March	668	.98	655	79	13.7	1.01	675	99	17.2
April	548	.95	521	−55	9.5	1.01	553	−23	−4.0
May	602	.92	554	−22	−3.8	.96	578	2	.3
June	565	.93	525	−51	−8.9	.93	525	−51	−8.9
July	605	.93	563	−13	−2.3	.96	581	5	.9
August	530	.92	488	−88	−15.3	.94	498	−78	−13.5
September	531	.95	504	−72	−12.5	.95	504	−72	−12.5
October	561	.95	533	−42	−7.3	.96	539	−37	−6.4
November	573	.97	556	−20	−3.5	.96	550	−26	−4.5
December	633	.94	595	19	3.3	.95	601	25	4.3
Station 502—ADT 4,868									
January	4,456	1.09	4,857	−411	−8.4	1.06	4,723	−145	−3.0
February	4,489	1.02	4,579	−289	−5.9	1.03	4,624	−244	−5.0
March	5,285	.98	5,179	311	6.4	1.01	5,338	470	9.7
April	5,736	.95	5,449	581	11.9	1.01	5,793	925	19.0
May	5,515	.92	5,074	206	4.2	.96	5,294	426	8.8
June	5,844	.93	5,435	567	11.6	.93	5,435	567	11.6
July	5,928	.93	5,513	645	13.2	.96	5,691	823	16.9
August		.92				.94			
September	5,099	.95	4,844	−24	−.5	.95	4,844	−24	−.5
October	5,140	.95	4,883	15	.3	.96	4,934	66	1.4
November	5,267	.97	5,109	241	4.9	.96	5,056	188	3.9
December		.94				.95			

Month	24-hour weekday volumes	Using mean Nashville factor: Factor	Estimated ADT	Error [1]: Volume	Error [1]: Percent	Using mean Memphis factor: Factor	Estimated ADT	Error [2]: Volume	Error [2]: Percent
Station 503—ADT 7,615									
January	7,436	1.09	8,105	490	6.4	1.06	7,882	267	3.5
February	7,834	1.02	7,991	376	4.9	1.03	8,069	454	6.0
March	7,967	.98	7,808	193	2.5	1.01	8,047	432	5.7
April	8,219	.95	7,808	193	2.5	1.01	8,301	686	9.0
May	8,641	.92	7,950	335	4.4	.96	8,295	680	8.9
June	8,010	.93	7,449	−166	−2.2	.93	7,449	−166	−2.2
July	8,295	.93	7,714	99	1.3	.96	7,963	348	4.6
August	8,453	.92	7,777	162	2.1	.94	7,946	331	4.3
September	8,363	.95	7,945	330	4.3	.95	7,945	330	4.3
October	7,861	.95	7,468	−147	−1.9	.96	7,547	−68	−.9
November	8,023	.97	7,782	167	2.2	.96	7,702	87	1.1
December	7,980	.94	7,501	−114	−1.5	.95	7,581	−34	−.4
Station 504—ADT 7,863									
January	8,061	1.09	8,786	923	11.7	1.06	8,545	682	8.7
February		1.02				1.03			
March	8,226	.98	8,061	198	2.5	1.01	8,308	445	5.7
April	8,734	.95	8,297	434	5.5	1.01	8,821	958	12.2
May	8,281	.92	7,619	−244	−3.1	.96	7,950	87	1.1
June	8,822	.93	8,204	341	4.3	.93	8,204	341	4.3
July	7,936	.93	7,380	−483	−6.1	.96	7,618	−245	−3.1
August	8,540	.92	7,857	−6	−.1	.94	8,028	165	2.1
September		.95				.95			
October	8,772	.95	8,333	470	6.0	.96	8,421	558	7.1
November	8,400	.97	8,148	285	3.6	.96	8,064	201	2.6
December	8,601	.94	8,085	222	2.8	.95	8,171	308	3.9
Station 505—ADT 17,439									
January	17,869	1.09	19,477	2,038	11.7	1.06	18,941	1,502	8.6
February		1.02				1.03			
March	17,420	.98	17,072	−367	−2.1	1.01	17,594	155	.9
April	17,061	.95	16,208	−1,231	−7.1	1.01	17,232	−207	−1.2
May	18,993	.92	17,474	35	.2	.96	18,233	794	4.6
June	19,157	.93	17,816	377	2.2	.93	17,816	377	2.2
July		.93				.96			
August	18,274	.92	16,812	−627	−3.6	.94	17,178	−261	−1.5
September	17,998	.95	17,098	−341	−2.0	.95	17,098	−341	−2.0
October	18,810	.95	17,869	430	2.5	.96	18,058	619	3.5
November	17,883	.97	17,346	−93	−.5	.96	17,168	−271	−1.6
December	19,789	.94	18,602	1,163	6.7	.95	18,800	1,361	7.8

[1] Standard deviation (Nashville): $S=\frac{\sqrt{\Sigma(\text{percent error})^2}}{N-1}=\frac{\sqrt{2,786.54}}{63-1}=\pm 6.7$ percent.

[2] Standard deviation (Memphis): $S=\frac{\sqrt{\Sigma(\text{percent error})^2}}{N-1}=\frac{\sqrt{3,255.18}}{63-1}=\pm 7.2$ percent.

Table 9.—Ratios (mean factors) of annual average daily traffic to average weekday traffic volumes for Nashville and Memphis, 1956

City and station number	January	February	March	April	May	June	July	August	September	October	November	December	Average daily traffic
Nashville:													
500	1.09	1.02	1.01	0.96	0.97	0.94	0.94	0.96	1.01	1.00	1.02	0.95	26,635
501	1.12	1.00	1.01	1.00	.95	1.04	1.01	1.01	1.00	.95	.97	.91	576
502	1.14	1.11	.94	.88	.89	.88	.84	.82	.93	.94	.96	.92	4,868
503	1.15	1.00	.94	.91	.89	.89	.92	.92	.93	.93	.96	.96	7,615
504	1.10	.98	.97	.94	.93	.94	.96	.88	.88	.88	.94	.94	7,863
505	1.03	.98	.98	.98	.91	.89	.90	.90	.94	.99	.98	.95	17,439
Mean	1.09	1.02	.98	.95	.92	.93	.93	.92	.95	.95	.97	.94	
Memphis:													
506	1.12	1.16	1.10	1.11	1.01	.95	.94	.90	.91	.90	.95	.99	23,671
507	.95	.99	.99	.97	.96	.87	.88	.93	.97	.97	.95	.99	10,394
508	1.05	1.06	1.02	.97	.95	.97	.98	.97	.98	.96	.97	.92	12,282
509	.99	.93	.95	.91	.90	.90	1.03	1.00	1.01	.99	1.01	.95	21,254
510	1.07	1.01	.98	1.01	.99	.96	.97	.95	.94	.95	.93	.92	7,058
511	1.17	1.04	1.04	1.10	.95	.90	.93	.87	.88	.96	.92	.95	27,903
Mean	1.06	1.03	1.01	1.01	.96	.93	.96	.94	.95	.96	.96	.95	

ıceptable as sufficiently accurate, then a 24-hour weekday traffic count may be assumed o represent the ADT.

Similar tests of Detroit and St. Louis data appear to bear out this conclusion with the following qualifications: (1) The months of January, July, August, and December show a high degree of dispersion for the test observations and hence are not representative months of the year; (2) there are low-volume roads in urban areas which will also show a high degree of dispersion and may not be reliable; and (3) the average weekday count is generally higher than the respective ADT, the average difference for the year being about +5 percent of the ADT. When seasonal variation is considered, the average range of the 24-hour weekday count is about 95 to 110 percent of the ADT. In Tennessee, because the factors are already available, the adjustments for monthly variations will be made.

Table 10.—Factors for the expansion of 4-hour urban counts to 24-hour counts on weekdays and the evaluation of the accuracy of these factors

Month	Number of counts	Average ratio of 24-hour traffic to 4-hour traffic			Number of counts	Average ratio of 24-hour traffic to 4-hour traffic		
		Ratio	Standard deviation of ratio	Standard error of ratio		Ratio	Standard deviation of ratio	Standard error of ratio
	6 a. m.–10 a. m.				7 a. m.–11 a. m.			
January	262	4.54	0.85	0.05	262	4.10	0.64	0.04
February	631	4.72	.84	.03	631	4.29	.62	.02
March	719	4.52	.73	.03	719	4.26	.55	.02
April	672	4.40	.59	.02	672	4.27	.52	.02
May	643	4.43	.55	.02	643	4.35	.51	.02
June	685	4.60	.63	.02	685	4.45	.58	.02
July	603	4.61	.70	.03	603	4.46	.59	.02
August	736	4.57	.67	.02	736	4.46	.61	.02
September	690	4.53	.84	.03	690	4.38	.67	.03
October	633	4.34	.66	.03	633	4.18	.57	.02
November	623	4.42	.72	.03	623	4.15	.60	.02
December	414	4.50	.73	.04	414	4.19	.60	.03
Average	--------	[1] 4.50	.71	------	--------	[1] 4.31	.59	------
	8 a. m.–12 noon				11 a. m.–3 p. m.			
January	262	4.53	0.65	0.04	262	4.47	0.54	0.03
February	631	4.50	.66	.03	631	4.26	.62	.02
March	719	4.50	.56	.02	719	4.30	.56	.02
April	672	4.53	.51	.02	672	4.39	.55	.02
May	643	4.60	.52	.02	643	4.50	.58	.02
June	685	4.54	.53	.02	685	4.33	.52	.02
July	603	4.59	.52	.02	603	4.47	.51	.02
August	736	1 63	.53	.02	736	4.46	.57	.02
September	690	4.57	.58	.02	690	4.51	.62	.02
October	633	4.45	.50	.02	633	4.46	.60	.02
November	623	4.39	.57	.02	623	4.34	.60	.02
December	414	4.45	.57	.03	414	4.35	.57	.03
Average	--------	[1] 4.53	.56	--------	--------	[1] 4.40	.57	--------
	12 noon–4 p. m.				1 p. m.–5 p. m.			
January	262	4.14	0.41	0.03	262	3.66	0.34	0.02
February	631	3.93	.48	.02	631	3.57	.36	.01
March	719	3.96	.45	.02	719	3.62	.33	.01
April	672	4.08	.42	.02	672	3.71	.10	.00
May	643	4.18	.48	.02	643	3.79	.34	.01
June	685	4.16	.42	.02	685	3.85	.10	.00
July	603	4.30	.45	.02	603	3.94	.36	.01
August	736	4.25	.51	.02	736	3.90	.37	.01
September	690	4.21	.52	.02	690	3.87	.39	.01
October	633	4.10	.49	.02	633	3.74	.37	.01
November	623	4.00	.47	.02	623	3.65	.34	.01
December	414	3.98	.48	.02	414	3.63	.34	.02
Average	--------	[1] 4.11	.47	--------	--------	[1] 3.75	.31	--------

[1] Weighted average based on card count.

Four-Hour Weekday Counts

Manual counts of 4-hour duration on weekdays are also used in Tennessee cities for the purpose of estimating ADT. The evaluation of the conversion of weekday 24-hour counts to estimates of ADT has already been discussed. Utilizing an electronic computer, a population study was made on the 1955 data of 33 urban continuous-count recorders for the purpose of determining and evaluating the procedure for the expansion of these 4-hour samples into estimates of traffic for 24-hour periods on weekdays.

Table 10 shows the mean expansion factors, the standard deviation, and the standard errors of the means of the expansion factors by months and by different 4-hour periods of traffic counts. The great similarities of the mean monthly factors and the consistency of the standard deviation for various 4-hour periods are evident. It is observed, however, that the greatest variation, average standard deviation ±0.71, occurs during the period from 6 a. m. to 10 a. m., being 15.8 percent of the mean factor of 4.5. The smallest variation is for the period from 1 p. m. to 5 p. m. for which the average standard deviation is ±0.31 or ±8.3 percent of the mean factor of 3.75. These characteristics indicate that estimates of 24-hour weekday volumes are accurate in terms of standard deviations of about ±12 to 13 percent, which may be considered satisfactory for practical purposes.

Traffic Article Postponed

The article *Traffic and Travel Trends*, which has appeared annually (except for 1954) in Public Roads magazine since 1946, will not be included this year.

The comprehensive study of highways, begun in 1956 in accordance with section 210 of the Highway Revenue Act of 1956, was given preference over the work of reporting and analyzing the 1957 traffic trends data. The consequent postponement of this work has delayed the publishing of the usual traffic trends article for this one year. Tabular material, which would have been a part of the report had it been published, will be available to subscribers of Public Roads during the first quarter of calendar year 1959, and a set of tables giving 1957 traffic information will be furnished at that time upon request addressed to the Bureau of Public Roads.

Traffic data furnished in conjunction with the section 210 study will undoubtedly result in revisions being made in tables to be published for 1957. Furthermore, the new information resulting from the section 210 study will make it possible to check a series of estimates of total rural and urban travel.

The present plan is to publish in Public Roads the 1957 traffic trends along with 1958 data when they become available. It is expected that the consolidated article will include a more detailed analysis of travel characteristics than has been published since the end of World War II.

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VOL. 30, NO. 6

FEBRUARY 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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BUREAU OF
PUBLIC ROADS
U. S. DEPARTMENT
OF COMMERCE
WASHINGTON

A Federal-aid secondary highway improvement in Marion County, Oreg. A sharply curved dog-leg (seen to the right) was eliminated by the new roadway.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 6 *February 1959*

C. M. Billingsley, *Editor*

BUREAU OF PUBLIC ROADS

Washington 25, D. C.

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IN THIS ISSUE

Progressive Alterations in a Sheet Asphalt Pavement Over a Long Period of Service 133

Airphoto Analysis of Terrain for Highway Location Studies in Maine 145

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

U. S. DEPARTMENT OF COMMERCE
LEWIS L. STRAUSS, Secretary

BUREAU OF PUBLIC ROADS
BERTRAM D. TALLAMY, Administrator
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Progressive Alterations in a Sheet Asphalt Pavement Over a Long Period of Service

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by JARL T. PAULS, Chief, Bituminous Branch, and WOODROW J. HALSTEAD, Head, Bituminous Materials Section

Progressive changes in the characteristics of the asphalt and the measurable physical properties of a sheet asphalt pavement constructed on upper Connecticut Avenue in Washington, D. C., were studied over a 19-year period. The tests conducted and reported in this article show that a large proportion of the hardening of the asphalt generally occurs during the mixing and rolling of the pavement. Although there was some indication that the extent and rate of change of properties differed with certain variations in the asphalts, the overall age-hardening in the pavement did not appear to be directly related to the type of asphalt used.

Both density of the pavement and Hubbard-Field stability of sample cores increased with time under the action of traffic, the rate of increase being greater in the early life of the pavement. The ultimate value obtained for the density varied with the lateral location of samples taken from the pavement. In the later years of the study, greatly increased traffic combined with the hardening of the asphalt resulted in the loss of a significant amount of the surface material.

THE tendency of asphalts to undergo alteration with accompanying deterioration of the pavement with age has been well established by laboratory studies and observations in the field. There have been few opportunities, however, for determining the rate, as well as the extent, of the alterations that occur under long-time service conditions and the relation of the changes in the asphalt to the measurable physical properties of the pavement. In order to obtain such information, a cooperative laboratory and field study was begun by the Bureau of Public Roads and the District of Columbia Department of Highways in 1935.[2]

During the ensuing period, periodic tests were made on samples taken from specific test sections of the pavement. It is the purpose of this article to present selected portions of the accumulated data that will show the progressive changes that have occurred in some of the more important properties of the asphalt and the pavement over a long period of time.

The experimental sections were planned to determine, if possible, (a) the effect of variations in certain characteristics of the asphalt on the service behavior of the pavement, (b) the effect of differences in volumes of traffic, (c) the changes in structural properties of the sheet asphalt surfacing with age, and (d) the changes that occur with age in the measured properties of the recovered asphalt.

General Description of Test Sections

Portions of two adjoining paving contracts on upper Connecticut Avenue in Washington, D. C., were selected as locations for the experimental sections. These contracts, which are designated in this article as contract 1 and contract 2, were awarded in 1935 to two local paving contractors. The location of the test sections and the general configuration of the pavement are illustrated in figure 1. The areas containing the A, B, and C test sections were part of contract 1, and the D sections were part of contract 2.

The contracts conformed in every respect to established standards for such work in the District of Columbia. During the laying of the test sections no attempt was made to change the contractors' methods of construction or their routine procedures. On both contracts the surfacing was laid in three lanes: two outer lanes each 20 feet wide and a center lane 19 feet wide. These lanes were separated by portland cement concrete headers, designed to serve as traffic guides. When completed, the elevation of the sheet asphalt pavement on each side of the headers was about ¼-inch higher than the headers.

The base for the center third of the street, previously occupied by streetcar tracks, was new portland cement concrete. In the two outer lanes the base was partly old and partly new concrete. In sections where the new surfacing was laid over old concrete, the sheet asphalt was laid directly against the face of the old concrete curb, so that the new surfacing along the curb served as a gutter for surface water. In sections where the old concrete in the outer lanes was replaced with a new concrete base, the sheet asphalt extended only to the edge of a new brick gutter, 12 inches wide. The surface course throughout was standard sheet asphalt. In the center third of the street the surface course was laid on a standard binder course having aggregate particles of ¾-inch maximum size. In the outer portions the binder course was of standard composition when laid on new cement concrete, and of the modified type when laid on old cement concrete. The aggregate in the leveling course or modified binder was ½-inch maximum size. The binder course was laid to about 1½ inches in compacted thickness as was also the sheet asphalt surfacing. Some variations from these thicknesses were found later in taking cores and other field samples. A summary of the information of a general nature pertaining to each contract follows.

Contract 1

Plant.—Stationary type using two driers, each 22 feet long and fired by 42-inch oil burners, and a 36-blade, 2,500-pound capacity, steam-jacketed, twin-pug mixer.

Rollers.—Two 10-ton steam tandems and one gasoline-driven tandem with front wheel diameters of 52 inches and rear wheel diameters of 40 inches.

Materials.—50–60 penetration asphalt was obtained from a local supplier; Potomac River sheet asphalt sand and limestone dust came from a commercial source.

Construction details.—Each load of mix was dumped on a metal plate, distributed with hand shovels, hand raked to grade, and finished with a 4-foot wooden lute. The steam rollers were used for initial compaction and the gasoline roller for transverse rolling and finishing.

Contract 2

Plant.—Stationary type using one 50-ton capacity oilburner drier and a 48-blade, 1,500-pound capacity, steam-jacketed, twin-pug mixer.

Rollers.—Two 10-ton gasoline tandems with front wheel diameters of 60 inches and rear wheel diameters of 49 inches.

Materials.—50–60 penetration asphalt was obtained from a local supplier (not the same as for contract 1). The Potomac River sheet asphalt sand and limestone dust came from the same commercial source as for contract 1.

Construction details.—The mixture was dumped on two metal plates, one on each

[1] This article was presented at the annual meeting of the Association of Asphalt Paving Technologists, Montreal, Canada, in February 1958. Mr. J. T. Pauls retired from the Federal service effective June 30, 1957.

[2] The helpful assistance of Messrs. H. F. Clemmer and N. G. Smith of the District of Columbia Department of Highways in planning and arranging for the various periodic inspections is gratefully acknowledged.

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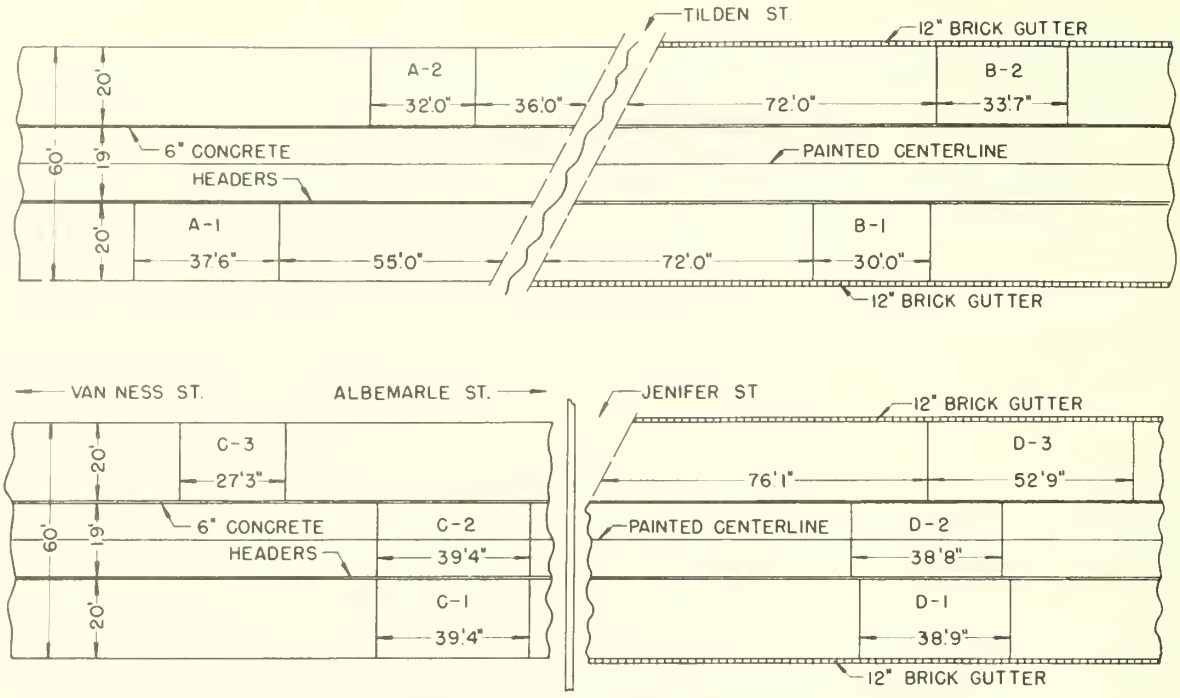

Figure 1.—Location of test sections on Connecticut Avenue in Washington, D. C.

side of a 2- by 4-inch longitudinal gage board placed in the center of each lane. The mixture was spread with shovels, raked to grade, and finished with a 6-foot wooden lute.

As illustrated in figure 1, the test sections designated as A–1, A–2, B–1, B–2, C–1, C–2, and C–3 were in the contract 1 area, and those designated as D–1, D–2, and D–3 were in the contract 2 area. Extensive data were obtained during the preparation of the sheet asphalt mixtures at the paving plant and during the spreading and rolling operations on the street. These data, in abridged form, are shown in table 1. The batch proportions shown in this table are the mix design. The variations indicated in the proportions of asphalt and filler are relatively small and were not considered of sufficient magnitude to significantly affect the properties of the mix.

The temperatures at the plant were recorded for each batch that went into the truckload which constituted the material for each test section. The individual temperatures generally varied about ±20° from the averages shown in the table.

Because of the difficulty in obtaining accurate readings, the temperature of the mixture was recorded in several locations as it was dumped and immediately before rolling. Where two values are shown, these are the minimum and maximum values. When one value is recorded, it is the average of four or five readings at different locations. The date on which each section was laid is indicated in the table and this accounts for the extent of the variations in the air temperature. All samples obtained following construction were taken in June or the early part of July, except for the final samples which were taken in April. Since sections A–1 and A–2 were laid in December 1935 and all the other sections were laid during the period from April 21 to July 7, 1936, the ages of the samples taken from sections A–1 and A–2 were recorded as ½ year greater than those for the other sections. Test sections A–1, A–2, C–1, and C–3 were laid on a leveling binder over old cement concrete, and were located in the outer lanes abutting the curb where they formed the gutters for the pavement. Sections D–1, D–3, B–1, and B–2 were laid on a standard binder over new cement concrete. They were in the outer lanes also, but they abutted a 12-inch-wide brick gutter. The other sections, C–2 and D–2, were in the center lane.

All asphalts used in contracts 1 and 2 came from two local suppliers and met the specification requirements. The asphalts used in contract 1 varied considerably in certain test characteristics, indicating that they may have been obtained from more than one source.

Summary of Findings

This investigation was planned to observe and study the effect of a number of important variables. However, all test sections gave good service and therefore the variations measured are within the range of good performance. In addition, the complexity of the interrelations between various factors makes it likely that any specific changes that occurred were the result of more than one cause. For these reasons specific conclusions cannot be drawn but a general summary is presented.

Table 1.—Mixture composition, temperatures, and other details of construction

Contract	Test section	Date of paving	Batch proportions, by weight			Average temperatures at plant			Average mixing time			Air temperature		Temperature of mix at paving location		Time rolled	Number of rollers
			Sand [1]	Filler [2]	Asphalt	Sand	Asphalt	Mix	Dry	Wet	Total	Minimum	Maximum	As dumped	Before rolling		
			Pct.	*Pct.*	*Pct.*	*°F.*	*°F.*	*°F.*	*Sec.*	*Sec.*	*Sec.*	*°F.*	*°F.*	*°F.*	*°F.*	*Min.*	
1	A–1	Dec. 10, 1935	78	11	11	----	----	----	----	----	----	53	56	320–350	160	65	2
	A–2	Dec. 20, 1935	78	11	11	400	290	340	18	67	85	----	30	240–310	190	60	2
	B–1	May 8, 1936	78	11	11	420	285	350	17	64	81	72	78	335	225	100	2
	B–2	May 27, 1936	79.5	10	10.5	420	270	355	16	67	83	76	88	340–350	260	140	3
	C–1	May 8, 1936	78	11	11	395	290	335	16	61	77	88	92	290	180	105	2
	C–2	Apr. 21, 1936	78	11	11	410	285	345	15	65	80	88	92	270–340	240	75	2
	C–3	May 26, 1936	79.5	10	10.5	400	280	355	16	64	80	88	90	340–350	220	80	3
2	D–1	June 17, 1936	79.8	9.7	10.5	480	290	380	0	64	64	----	89	305	240	90	2
	D–2	May 28, 1936	79	10	11	390	290	335	0	66	66	68	74	300–330	240–280	145	2
	D–3 [3]	July 7, 1936	79.8	9.7	10.5	390	290	335	0	61	61	82	86	270–320	190–200	175	2

[1] Sand contained an average of 3.4 percent passing the No. 200 sieve.
[2] Commercial limestone filler was added.
[3] Two truckloads were used on this section; other sections required only one load each.

able 2.—Mechanical analysis of sand and filler sampled before incorporation in the mixture

Sieve sizes	Contract 1 test sections							Contract 2 test sections		
	A-1	A-2	B-1	B-2	C-1	C-2	C-3	D-1	D-2	D-3
Percentage (by weight) of sand passing—										
No. 10 sieve	100	100	100	100	100	100	100	100	100	100
No. 20 sieve	98	99	99	98	98	99	98	99	96	98
No. 30 sieve	93	94	95	93	93	95	91	95	90	92
No. 40 sieve	82	82	85	80	82	85	77	82	78	80
No. 50 sieve	63	60	66	59	64	67	56	57	59	62
No. 80 sieve	35	27	38	33	38	37	32	28	36	34
No. 100 sieve	25	18	27	23	27	27	23	19	28	25
No. 200 sieve	4	3	3	3	3	4	3	2	5	4
Percentage (by weight) of commercial filler passing No. 200 sieve	95	96	95	95	96	97	95	95	95	94

Changes in the asphalt.—The asphalts used were from two suppliers but the long period of time over which the construction was carried out creates an uncertainty as to whether all the asphalts from the same supplier were from the same source and method of refining. The characteristics of the materials used in contract 1 indicate differences that are greater than would normally be expected for materials from the same source and method of manufacture. However, these asphalts met the specification requirements, and there is no evidence of failure that can be attributed solely to the characteristics of the materials.

All asphalts showed considerable change in properties during the mixing and laying of the pavement and during the 19 years of service. A large percentage of this change took place prior to compaction of the mix on the street. The changes in penetration and softening points during the mixing cycle were of the same order for all asphalts. However, changes in ductility and the amount of organic insoluble in 86° B. naptha were generally larger for the asphalts used in contract 1 than for those used in contract 2. The changes that occurred in the asphalt while in the pavement did not appear to be directly related to any of the characteristics of the asphalt and the degree of change was evidently affected to a considerable extent by traffic conditions, location in the pavement, and other localized conditions.

Limited studies after 19 years' service showed considerable variation in the hardness of the recovered asphalts within the same section. It was found that the asphalt in the portion of the pavement near the surface hardened to a greater extent and lost a larger proportion of its ductility than did the material in the center or bottom portions of the pavement.

The hardness of the asphalts in some of these sections approached values that have been reported elsewhere as being critical and sufficient to cause disintegration of the surface. Except in locations where the portland cement concrete base had failed, these pavements continued to give good service. This may have resulted from the inflexibility of the base which limited movement in the surface. It is quite possible that similar hardening in the surface placed over a more flexible base would have resulted in failure.

However, it should be pointed out that similarly constructed sheet asphalt pavements in the District of Columbia in locations with smaller traffic volumes have shown severe cracking and deterioration at ages for which the test sections were still in good condition. Therefore it is probable that factors not isolated by these tests have a marked effect on the tendency of the pavement to crack.

Changes in mineral aggregate.—Degradation of the sand aggregate did not occur to any appreciable extent.

Changes in pavement characteristics.—The density of the pavement after the initial rolling was generally lower than was considered the best construction practice, but there was no evidence that this condition affected the performance of the pavement. Compaction of the sheet asphalt surfacing continued under traffic, and a large percentage of the total compaction took place during the first year of service. The density of the pavement varied with the lateral location; the portions of the pavement near the curbs and the concrete headers generally had lower densities than the portions near the center of the various sections.

Stability of the sheet asphalt surfacing, as measured by Hubbard-Field tests on cores taken from the pavement, increased progressively with age and densification of the pavement for about 4 years. Results on cores taken after 10 years were inconclusive because of the inherent difficulties in obtaining satisfactory tests.

Because of the variations in densities obtained at different locations in each section at various times, the specific relation between the void content of the pavement and that obtained by laboratory design procedures was not definitely established, but a comparison of the minimum voids obtained for any group of samples showed that the values calculated from laboratory compaction approached the minimum values obtained in the road after one year under traffic.

Thickness measurements indicated that a considerable percentage of the surfacing was worn away after 19 years. There was some evidence that the amount of material lost from the surface of the pavement by the action of traffic varied with the hardness of the asphalt, but since the center lanes which normally carry the smaller amounts of traffic generally showed the least asphalt hardening, no specific relation was established.

Miscellaneous factor.—Although not a direct factor in this investigation, the performance of brick gutters was greatly superior to gutters constructed with sheet asphalt. Failure of the latter occurred in less than 10 years, whereas the brick gutters were still in good condition after more than 19 years' service.

Table 3.—Bitumen content and grading of the aggregate by extraction tests

Test section	Source of sample	Year	Bitumen extracted, percent	Grading of extracted aggregate, percentage passing sieve sizes—							
				No. 10	No. 20	No. 30	No. 40	No. 50	No. 80	No. 100	No. 200
A-1	Loose mix	1935	10.6	100	97	91	79	59	35	28	15
	Cores	1937	10.7	100	97	91	79	60	38	31	17
	do	1940	11.7	100	97	91	79	61	39	32	17
	do	1946	10.3	100	98	92	79	60	38	31	17
	Cut samples	1955	11.1								
A-2	Loose mix	1935	10.7	100	97	91	78	58	33	27	15
	Cores	1937	10.7	100	98	92	80	60	37	31	19
	do	1940	11.2	100	97	91	78	58	34	28	15
	do	1946	10.8	100	98	92	79	60	37	31	19
B-1	Loose mix	1936	11.0	100	98	93	82	63	40	32	14
	Cores	1940	11.0	100	98	93	82	64	43	34	15
	do	1946	11.3	100	98	93	81	63	43	35	17
	Cut samples	1955	11.0								
B-2	Loose mix	1936	10.7	100	98	92	81	63	42	34	14
	Cores	1940	10.6	100	98	93	82	65	45	36	17
	do	1946	10.5	100	98	92	80	63	43	35	16
	Cut samples	1955	11.0								
C-1	Loose mix	1936	10.7	100	97	92	81	64	42	34	14
	Cores	1940	10.0	100	98	92	80	63	43	34	14
	do	1946	10.7	100	98	91	79	62	43	35	15
C-2	Loose mix	1936	10.9	100	98	92	81	62	36	29	14
	Cores	1940	10.7	100	97	91	78	60	37	29	14
	do	1946	10.7	100	98	92	80	61	39	32	15
C-3	Loose mix	1936	10.3	100	97	90	78	60	40	33	15
	Cores	1946	10.2	100	97	91	78	60	42	35	15
D-1	Loose mix	1936	11.1	100	99	95	84	61	34	26	11
	Cores	1940	10.5	100	98	94	81	59	35	27	12
	do	1946	10.9	100	98	94	79	57	34	26	11
	Cut samples	1955	11.3								
D-2	Loose mix	1936	10.9	100	97	92	81	65	44	36	15
	Cores	1940	11.1	100	97	92	81	65	45	36	15
	do	1946	10.9	100	98	92	81	65	46	38	16
D-3	Loose mix	1936	10.3	100	97	91	80	64	42	34	16
	Cores	1940	9.1	100	97	91	79	63	42	33	15
	do	1946	10.5	100	98	92	80	64	43	35	16

This may have been caused by a poor seal between the concrete of the curb and the sheet asphalt, which permitted water to get in and under the surface with subsequent acceleration of deterioration by freezing.

Sampling and Testing Procedure

In order to accomplish the stated objectives of the project, individual samples of all the various materials used in each contract and samples of freshly prepared uncompacted sheet asphalt mixtures were obtained at the paving plant and sent to the laboratory for testing. After the surfacing had been compacted and before opening the street to traffic, 2-inch diameter cores were cut from each experimental section except B–1, where it was impractical to close the street. These cores were taken from the pavement in pairs about 6 inches apart longitudinally and at 2-foot intervals across the width of the section. Cores were similarly cut at various times throughout a 10-year period. At the end of 19 years, final samples were cut as slabs by air hammers.

The program of laboratory tests was based on obtaining as complete information as possible on the original materials, especially the asphalts, and then determining the changes that occurred in the various measurable properties of both the asphalt and the pavement mixture.

Tests made on the original asphalt included penetration tests at various temperatures, ductility tests at various temperatures and speeds, softening points, solubilities in both carbon tetrachloride and 86° B. naphtha, naphtha-xylene equivalents, and special thin-film oven tests on two representative samples. The studies of the mixes and cored samples were based on the determinations of the density (apparent specific gravity) of the top inch of each core as it was taken from the road and Hubbard-Field stability of both the mixes compacted in the laboratory and the cored samples. The asphalt was recovered by the Abson method for the determination of the various properties for comparison with those of the original asphalt. Bitumen content was determined by extraction of the field samples, and a portion of the aggregate extracted from each sample was graded for comparison with the original.

Typical analyses of sand and filler sampled before incorporation in the mixture are given in table 2. The gradation of the sand sampled from the stock piles at the plants was reasonably uniform. Such variations as are shown in the table are not believed to be significant. The filler was limestone dust and was quite uniform with respect to the percentage passing the 200-mesh sieve. The apparent specific gravity of the sand was 2.67 and that of the filler was 2.70.

Results of Extraction and Recovery Tests

Results of sieve analysis of the mineral aggregate, extracted from samples of the original mixtures taken during construction of the sections and from samples taken from the pavement at various times, are shown in table 3. Also given in this table are the percentages of bitumen extracted from the samples. The results reported for the co are those obtained on composite samples co sisting of the top inch of all cores taken from particular test section at any one time, and a not those for single cores. For the cut sampl taken in 1955, the results given in table for sections A–1 and B–2 were obtained fro single samples, whereas those for sectio B–1 and D–1 were from a composite sampl

In all sections the average amount of bit men extracted from the cut samples agre closely with the amount previously shown table 1. However, in two sections, A–1 a D–3, the range between maximum and mi imum content of extracted bitumen is 1 percent. This spread greatly exceeds t range of asphalt content found in the oth sections which have an average spread of on 0.4 percent, a maximum of 0.8 and a minimu of 0.1 percent. Variations, such as shown sections A–1 and D–3, could possibly affe the service behavior of the pavement b generally for most sections the variatio are within the normal range to be expected routine plant operations.

Grading of the aggregate, also shown table 3, indicates that no appreciable amou of degradation occurred during the 10-ye period of traffic. The specific gravity composite aggregate extracted from the co samples was 2.67.

Table 4 gives the results of tests on (a samples of the asphalt taken before bei incorporated into the mixture; (b) sampl of the asphalt taken before being incorp rated into the mixture, dissolved in benzen and then recovered; and (c) bitumen extract

Table 4.—Results of tests on the original asphalt, the original asphalt fluxed with solvent and recovered, and the asphalt extracte from mixes sampled prior to compaction

Test section	Source of asphalt	Specific gravity at 77°/77° F.	Penetration					Softening point	Ductility		Insoluble in CCl_4		Organic insoluble in 86° B. naphtha	Naphtha-xylene equiva-lent
			100 grams, 5 seconds				200 grams, 60 seconds, at 32° F.		At 77° F., 5 cm. per min.	At 39.2° F., ¼ cm. per min.	Organic	Inorganic		
			At 95° F.	At 77° F.	At 59° F.	At 32° F.								
								°F.	*Cm.*	*Cm.*	*Percent*	*Percent*	*Percent*	*Percent*
A–1	Original asphalt from storage tank	1.030	148	58	24	5	14	125	155	8.0	0.04	0.15	24.4	16–20
	Original, fluxed and recovered		142	58	24	5	14	128	118	7.5	.09	.08	24.5	16–20
	Extracted from mixture		68	31	13	3	12	143	29	4.3	.06	.0	29.2	20–24
A–2	Original asphalt from storage tank	1.032	145	58	25	5	14	126	203	7.3	.02	.08	24.9	16–20
	Original, fluxed and recovered		148	60	24	5	14	128	138	7.3	.07	.12	25.2	16–20
	Extracted from mixture		70	34	14	3	11	143	40	4.5	.03	.0	28.6	20–24
B–1	Original asphalt from storage tank	1.026	144	56	24	3	17	127	226	14.3	.13	.0	23.4	0–4
	Original, fluxed and recovered		139	55	23	3	18	128	229	15.5	.14	.0	24.5	4–8
	Extracted from mixture		73	33	14	2	11	141	80	5.3	.15	.0	27.6	4–8
B–2	Original asphalt from storage tank	1.032	134	53	23	3	17	129	233	13.0	.08	.04	25.2	4–8
	Original, fluxed and recovered		124	53	23	5	15	130	237	11.5	.12	.0	25.4	8–12
	Extracted from mixture		68	33	15	4	12	145	36	4.8	.09	.04	28.9	12–16
C–1	Original asphalt from storage tank	1.028	138	56	22	4	15	126	205	13.3	.09	.01	23.1	0–4
	Original, fluxed and recovered		139	57	22	4	15	127	216	13.5	.10	.0	23.7	0–4
	Extracted from mixture		70	33	14	3	11	144	35	4.8	.05	.04	28.9	0–4
C–2	Original asphalt from storage tank	1.027	140	55	22	4	19	126	250+	14.0	.11	.06	23.2	0–4
	Original, fluxed and recovered		123	51	22	4	15	130	250+	10.5	.15	.0	24.3	4–8
	Extracted from mixture		75	36	14	3	12	142	39	5.0	.07	.04	27.9	8–12
C–3	Original asphalt from storage tank	1.030	136	55	23	4	19	127	250+	12.5	.24	.0	24.3	4–8
	Original, fluxed and recovered		129	54	23	5	17	130	195	9.5	.14	.01	25.5	8–12
	Extracted from mixture		67	32	14	2	10	145	33	4.8	.06	.0	28.7	12–16
D–1	Original asphalt from storage tank	1.024	148	57	24	4	15	126	244	14.8	.08	.02	23.0	0
	Original, fluxed and recovered		152	60	24	3	20	127	239	13.3	.14	.0	23.3	0
	Extracted from mixture		72	32	14	2	11	143	105	5.5	.06	.0	26.9	0
D–2	Original asphalt from storage tank	1.022	144	56	24	3	12	126	250	15.5	.10	.0	21.8	0
	Original, fluxed and recovered		130	52	22	3	14	128	248	11.0	.16	.01	23.1	0
	Extracted from mixture		77	34	15	2	11	139	141	5.3	.14	.0	25.2	0
D–3	Original asphalt from storage tank	1.022	150	58	22	4	15	126	250+	17.8	.08	.03	22.2	0
	Original, fluxed and recovered		139	55	22	4	18	128	250+	14.0	.14	.0	23.1	0
	Extracted from mixture		86	36	15	3	10	137	141	6.0	.08	.0	25.1	0

ble 5.—Results of thin-film oven tests on representative asphalts

Film thickness	Time of heating	Tests on residue: Penetration at 77° F.	Softening point	Ductility at 77° F.	Retained penetration
A.—Material Used in Section A–2, Contract 1					
In.	*Hr.*		*° F.*	*Cm.*	*Pct.*
----	[1] 0	58	126	203	------
1/8	2	42	136	118	72
1/8	5	32	145	29	55
1/8	7	28	151	14	48
1/16	2	37	141	63	58
1/16	5	26	155	8	45
1/16	7	21	163	6	36
1/32	2	30	150	15	52
1/32	5	18	176	4	31
1/32	7	16	185	3	27
B.—Material Used in Section D–2, Contract 2					
----	[1] 0	56	126	250	------
1/8	5	33	139	155	59
1/16	5	26	147	35	46

[1] Original asphalt.

om loose mixture samples and recovered. ests on the original asphalt, dissolved and ecovered, show the effect of the recovery rocess on the several test characteristics of ie asphalts, whereas the tests on the extracted nd recovered bitumen from the mix show the ffect of the mixing process on these charac- eristics.

Considering, first, the characteristics of ie original asphalts sampled from the paving lant storage tanks, the results of the pene- ration test at 77° F. and the softening point etermination for the asphalts from all ections in both contracts were in close greement.

For the asphalts used in contract 1, there ere variations in some of the properties that ere greater than would normally be expected or asphalts manufactured from the same ource by the same method of refining. As hown in table 1, these asphalts were supplied ver a considerable period of time, therefore t is quite possible that some changes in the rude material or method of refining occurred. pecific information relative to this possibility s not available. However, the specific ravity at 77° F., the ductility at 39.2° F. 1/4 cm. per minute), the naphtha insoluble, nd the naphtha-xylene equivalent values iven in table 4 indicate that the asphalts sed for sections A–1 and A–2 in December 1935 were similar.

The asphalts used during the period from April 21 to May 8, 1936, for sections C–2, B–1, and C–1 were similar, and likewise the material used May 26 and 27 for sections C–3 and B–2. The test results for the asphalts used in all three sections of contract 2 showed a high degree of uniformity in all test characteristics.

The results given in table 4 show that marked alteration occurred in the asphalts during the mixing and spreading processes. Comparison of the test results of the asphalts extracted and recovered from the samples of loose mixtures with corresponding results of tests on the original asphalts shows pronounced losses in penetration and ductility, and increases in softening point temperatures. The percentage of retained penetration at 77° F. ranged from 71 percent of the initial value in section C–2 to 53 percent in sections A–1 and D–1, and averaged 60 for all sections. Ductilities at 77° F. were reduced to values ranging from 29 cm. in section A–1 to 141 cm. in sections D–2 and D–3. Increases in softening point temperature ranged from a minimum of 9° F. in section D–3 to a maximum of 17 degrees in section C–1.

The changes in penetration and softening points were not significantly different for the asphalts used in contract 1, when compared with the corresponding changes in asphalts used in contract 2. However, the decreases in ductility at 77° F., 5 cm./min., for the asphalts used in contract 1 are generally greater than the corresponding decreases for materials used in contract 2. There were also greater increases in the average amounts of organic insoluble in 86° B. naphtha for contract 1 asphalts than for contract 2 asphalts.

Table 5 shows the results of special thin-film oven tests conducted in order to obtain further information on the possible differences in asphalts used in contracts 1 and 2 and to evaluate their susceptibility to change under controlled conditions. The material designated as *A* was that used in section A–2 and that designated as *B* was used in section D–2. These tests were made by heating 50-ml. (volume at 77° F.) samples of each asphalt in an oven at 325° F. in different size containers so that various film thicknesses were obtained. Various heating periods were used for each film thickness. The residues obtained for each condition of test were examined for penetration, ductility, and softening point.

Studies of the relation of the amount of hardening that occurs in the thin-film oven test have shown that the losses in penetration at 77° F. and the ductility at 77° F., 5 cm./min., for a film thickness of 1/8 inch and a heating time of 5 hours, generally approximate the losses occurring in the plant mixing and laying operation (*1*, *2*).[3] Sample *B* which was used in section D–2 showed higher resistance to change than did sample *A* which was used in section A–2, contract 1. The 1/8-inch, 5-hour residue for *B* retained 59 percent of its original penetration and had a ductility of 155 cm. at 77° F. The same residue for *A* retained 55 percent of its original

[3] Italic numbers in parentheses refer to the list of references on p. 144.

Table 6.—Characteristics of asphalts recovered from the pavements at various ages

Test section	Age of pavement	Penetration at 77° F., 100 g., 5 sec.	Softening point	Ductility: At 77° F., 5 cm./min.	Ductility: At 39.2° F., 1/4 cm./min.	Organic insoluble in 86° B. naphtha	Naphtha-xylene equivalent	PTS[1]
	Years		*° F.*	*Cm.*	*Cm.*			
A-1	[2] 0	31	143	29	4.3	29.2	20–24	0.0214
	2½	32	144	26	4.7	29.7	20–24	.0209
	4½	28	146	20	4.0	29.1	24–28	.0211
	10½	24	154	9	2.3	30.6	28–32	.0198
	19½	22	156	7	---	31.3	16–20	.0198
A-2	[2] 0	34	143	40	4.5	28.6	20–24	.0208
	2½	38	139	100	5.3	29.0	20–24	.0213
	4½	31	142	52	4.8	28.0	24–28	.0217
	10½	27	148	15	3.8	30.7	28–32	.0207
B-1	[2] 0	33	141	80	5.3	27.6	4–8	.0216
	2	29	145	30	4.3	28.8	4–8	.0212
	4	32	143	66	4.5	29.2	4–8	.0212
	10	25	151	14	3.3	30.1	8–12	.0203
	19	14	166	5	---	---	---	.0197
B-2	[2] 0	33	145	36	4.8	28.9	12–16	.0204
	2	26	151	12	2.0	31.8	8–12	.0201
	4	18	158	6	.5	32.1	16–20	.0203
	10	17	164	5	.3	34.6	20–24	.0192
	19	13	171	4	---	34.0	12–16	.0190
C-1	[2] 0	33	144	35	4.8	28.9	0–4	.0221
	2	29	145	28	4.4	29.0	8–12	.0212
	4	25	146	24	4.5	28.0	8–12	.0218
	10	24	152	12	3.5	30.3	12–16	.0203
C-2	[2] 0	36	142	39	5.0	27.9	8–12	.0207
	2	30	146	36	4.5	28.4	4–8	.0207
	4	28	145	25	4.3	28.2	12–16	.0214
	10	31	145	41	4.5	29.1	12–16	.0208
C-3	[2] 0	32	145	33	4.8	28.7	12–16	.0206
	2	26	150	14	2.8	30.9	12–16	.0204
	4	23	154	8	2.0	31.8	20–24	.0200
	10	18	162	5	0	33.9	20–24	.0194
D-1	[2] 0	32	143	105	5.5	26.9	0	.0212
	2	29	146	23	4.5	28.9	0	.0209
	4	33	144	38	4.5	27.7	0	.0207
	10	26	150	18	1.8	29.1	0–4	.0204
	19	21	157	8	---	---	---	.0198
D-2	[2] 0	34	139	141	5.3	25.2	0	.0221
	2	33	141	62	5.0	26.8	0	.0216
	4	34	141	68	5.0	27.1	0	.0214
	10	32	141	167	5.3	26.3	0–4	.0218
	19	31	144	69	---	26.9	---	.0211
D-3	[2] 0	36	137	141	6.0	25.1	0	.0224
	2	31	141	106	5.2	27.0	0	.0221
	4	32	142	114	5.0	26.7	0–4	.0215
	10	23	148	23	3.5	28.4	8–12	.0217
	19	21	153	10	---	27.6	0	.0208

[1] Penetration-temperature-susceptibility factor.
[2] Samples of loose mixtures at time of construction.

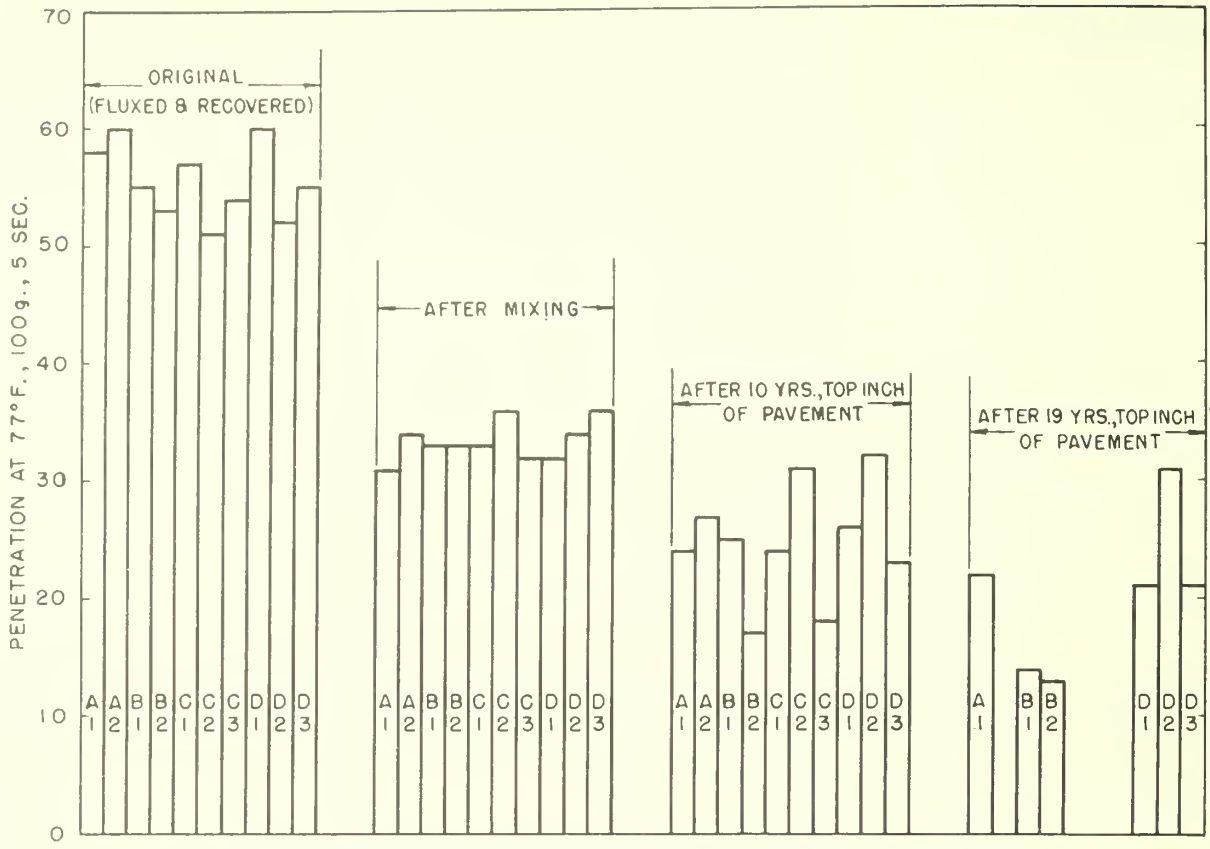
ORIGINAL
(FLUXED & RECOVERED)
AFTER MIXING
AFTER 10 YRS., TOP INCH OF PAVEMENT
AFTER 19 YRS., TOP INCH OF PAVEMENT
PENETRATION AT 77°F., 100g., 5 SEC.
0
10
20
30
40
50
60
70
A 1 A 2 B 1 B 2 C 1 C 2 C 3 D 1 D 2 D 3
A 1 A 2 B 1 B 2 C 1 C 2 C 3 D 1 D 2 D 3
A 1 A 2 B 1 B 2 C 1 C 2 C 3 D 1 D 2 D 3
A 1 B 1 B 2 D 1 D 2 D 3

Figure 2.—Penetration of recovered asphalts at various time periods.

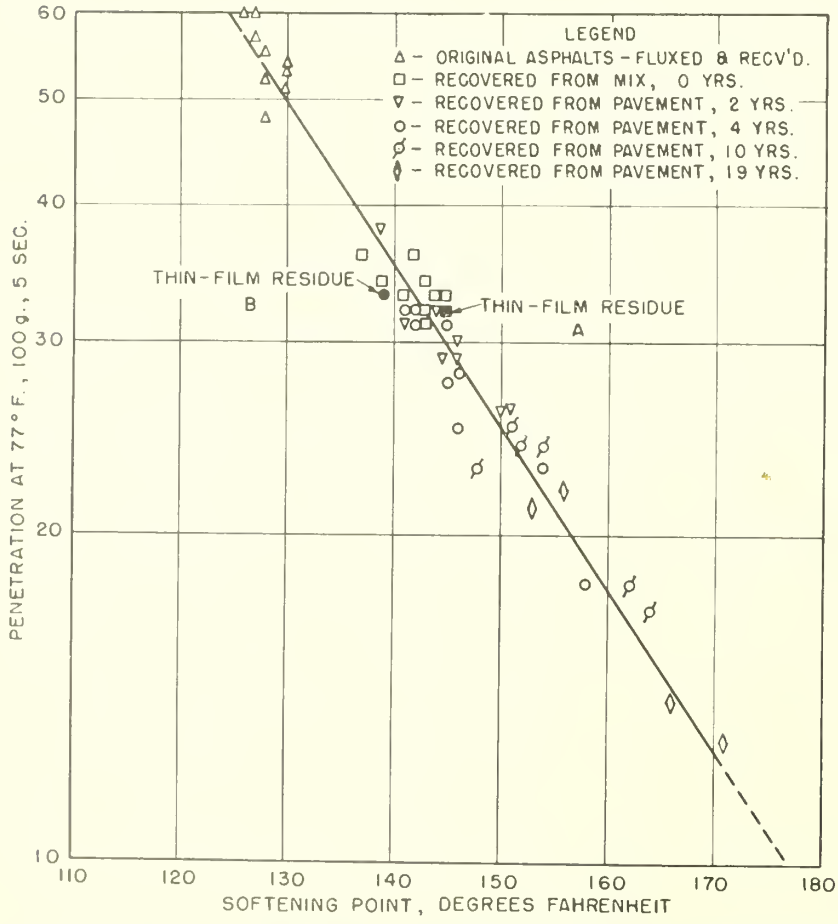

Figure 3.—Penetration-softening point relation for asphalts recovered from pavement test sections.

penetration but had a ductility of only 29 cm. Both of the original asphalts had ductilities greater than 200 cm. It should be borne in mind that these results may not necessarily be typical of the materials actually used in all sections of contract 1 because of the variations in the asphalts.

Further Alterations in Asphalt Characteristics With Age

The effect of age on certain test characteristics of the asphalts extracted from cores and cut pavement samples taken at time intervals over a period of 19 years is shown in table 6. The test results of the bitumen extracted from samples of loose mixtures are shown as the zero age of pavement for each section. These results are taken as the basis for comparison of the progressive alteration of the asphalts in the various sections.

Table 6 shows the progressive hardening of the asphalts that occurred with age. This is evidenced by lower penetration and ductility and by higher softening point temperatures. In addition to these physical changes, some chemical changes also occurred during aging as evidenced by the general trend toward increased percentages of organic material insoluble in 86° B. naphtha, increased naphtha-xylene equivalents, and a decrease in the penetration-temperature-susceptibility factor. The penetration-temperature-susceptibility factor was calculated using the hypothesis of J. Ph. Pfeiffer and P. M. Van Doormal (*3*) that the penetration of an asphalt at the softening point is equal to 800.

$$PTS=\frac{\log 800-\log p}{t-77}$$

Where:

PTS = Penetration-temperature-susceptibility factor.

p = Penetration at 77° F., 100 g., 5 sec.

t = Softening point, °F.

Figure 2 provides a comparison of the penetration of the recovered asphalts after mixing and after the several periods of aging of the original asphalt (fluxed and recovered). This graph shows that a large proportion of the change in penetration occurs during the mixing process, being 50 percent or more of the total drop in penetration over the 19-year period for which recoveries were made. Figure 2 also reflects a considerable range in penetration values for the various sections. The recovered asphalt from section D–2 after 19 years had a penetration of 31 at 77° F., the highest for any of the sections for which data were available, and the asphalt from section B–2 after 19 years had a penetration of 13 at 77° F., the lowest obtained.

In order to obtain a more complete picture of the alterations in the properties of the asphalt, in addition to the penetration, the softening points, ductilities, and the relations between these characteristics and the penetrations must also be considered. The penetration-softening point relations are shown in figure 3 in which the penetration of each original and recovered asphalt is plotted on a logarithmic scale against its softening point. Although there is some scatter of the points, a good straight line relation exists. This relation indicates changes that occurred in the penetration-temperature-susceptibility factor were directly related to the change in penetration, and there was no general trend toward an increased degree of oxidization as hardening continued.

Figure 4 shows the logarithm of the ductility values plotted against the penetration. Although two general curves have been drawn to show the differences between asphalts used in contract 1 and those used in contract 2, it is recognized that for penetration values of 30 or below the differences between asphalts from contracts 1 and 2 are no greater than the differences between various asphalts recovered from the same project. For penetration values of 31 or greater, the ductility-penetration relation for the materials from the two contracts appears to be significantly different. The softer materials recovered from contract 2 have considerably more ductility for the same penetration than do the materials recovered from contract 1.

Pavement Changes Related to Increases in Softening Point

Investigations reported by Lewis and Welborn (*4*) show that most of the physical properties of bituminous mixtures vary directly with the logarithm of the penetration of the asphalt rather than with the penetration itself. This means that the effects of equal increments of change in penetration progressively increase as the material becomes harder.

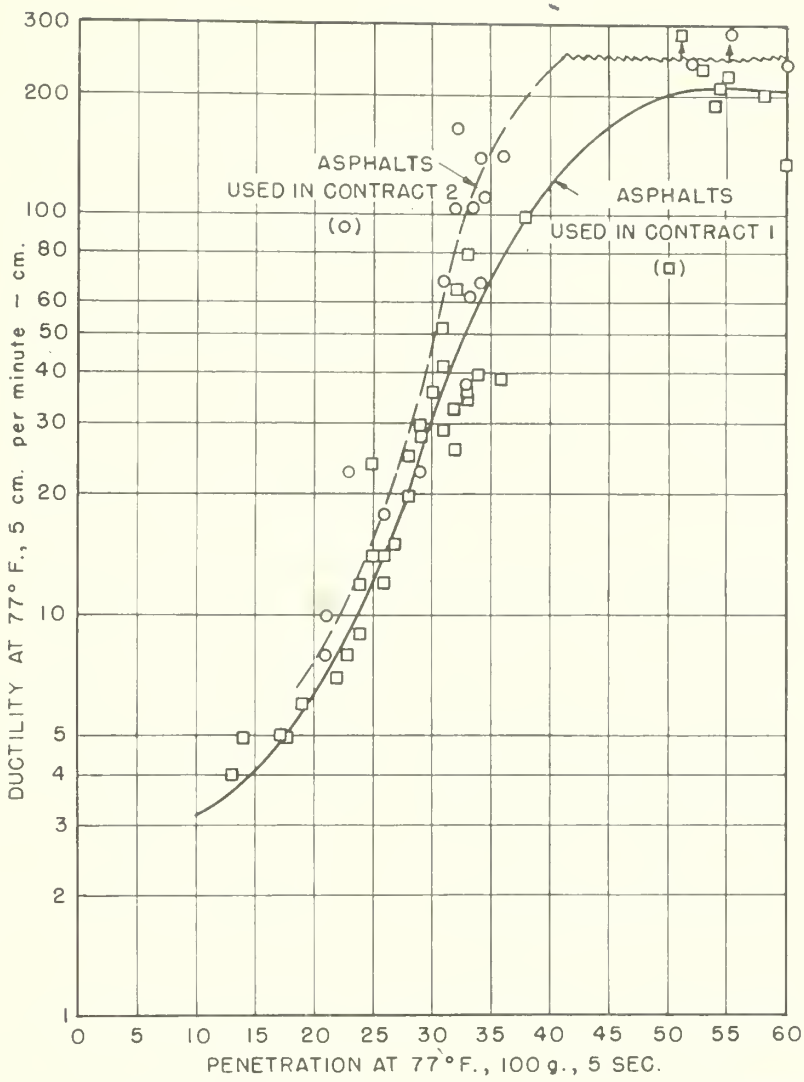

Figure 4.—Ductility-penetration relation for asphalts recovered from pavement test sections.

Figure 3 shows that the softening point also varies as the logarithm of the penetration; thus the softening point increase provides a measure of the hardening of the asphalt that is more directly related to the change in the properties of the pavement than is a consideration of penetration changes only.

A convenient relative index of change that is related to the percentage of total hardening or alteration that can be expected during the lifetime of the pavement can be calculated from the softening point data. Such an index is based on the arbitrary selection of a softening point that is representative of zero hardening and one that is representative of 100 percent hardening. The logical point of zero hardening for any single material is the softening point of the original asphalt, but in order to place all the materials in this series on the same basis a value of 127° F. was chosen. This value is shown by the curve in figure 3 for 55 penetration, the midpoint of the 50–60 specification grade. It also closely approximates the average softening point of all original asphalts which was 126.4° F.

To select a point representative of 100 percent hardening, earlier research studies were reviewed. One frequent observation was that many pavements deteriorate rapidly when the penetration of the asphalt reaches a critical point. This critical value has been variously estimated from 10 to 20 penetration at 77° F., depending somewhat on climatic conditions (*4*). Table 6 shows that the lowest penetration, at 77° F., of the recovered asphalts of the series is 13. Therefore, a penetration value of 10 appears to be a suitable reference for the selection of the point of 100 percent hardening. When the softening-point, log-penetration curve of figure 3 is extrapolated, the softening point for asphalts with a penetration of 10 is 177° F.

Thus, the total change in softening point that is expected for these asphalts during the lifetime of the pavement is 50° F. (177°–127°), and the hardness index of any particular recovered asphalt is the proportion of this total change that has already occurred. This can be expressed as a general algebraic relation as follows:

$$\text{Hardness index}=\frac{SP_x-SP_1}{SP_2-SP_1}$$

Where:

SP_x = Softening point of the recovered asphalt being considered.

SP_1 = Softening point of the chosen zero hardening point (127° F. for these asphalts).

SP_2 = Softening point of the chosen 100 percent hardening point (177° F. for these asphalts).

Table 7.—Relative hardness of asphalts at various times based on increases in softening point

Comparison	Hardness index [1] for materials from test sections—									
	A-1	A-2	B-1	B-2	C-1	C-2	C-3	D-1	D-2	D-3
Original asphalt	−4	−2	0	4	−2	−2	0	−2	−2	−2
Original asphalt recovered [2]	2	2	2	6	0	6	6	0	2	2
Thin-film residue [3]	---	---	---	---	36	---	---	---	24	---
Asphalt from mix [4]	32	32	28	36	34	30	36	32	24	20
Asphalt from pavement [4] at—										
2 years	---	---	36	48	36	38	46	38	28	28
2½ years	34	24	---	---	---	---	---	---	---	---
4 years	---	---	32	62	38	36	54	34	28	30
4½ years	38	30	---	---	---	---	---	---	---	---
10 years	---	---	48	72	50	36	70	46	28	42
10½ years	54	42	---	---	---	---	---	---	---	---
19 years	---	---	78	88	---	---	---	60	34	52
19½ years	58	---	---	---	---	---	---	---	---	---
Hardening during mixing	30	30	26	30	34	24	30	32	22	18
Hardening in pavement at—										
10 years	[5] 22	[5] 10	20	36	16	6	34	14	4	22
19 years	[6] 26	---	50	52	---	---	---	28	10	32
Total hardening (mix plus service):										
10 years	[5] 52	[5] 40	46	66	50	30	64	46	26	40
19 years	[6] 56	---	76	82	---	---	---	60	32	50

[1] Softening point of 127° F. (equivalent to penetration of 55) taken as zero hardness index; softening point of 177° F. (equivalent to a penetration of 10) taken as 100 hardening index.
[2] Original asphalt dissolved in benzol and recovered by Abson method.
[3] Residue from ⅛-inch film heated 5 hours at 325° F.
[4] Extracted with benzol and recovered by Abson method. [5] Value for 10½ years. [6] Value for 19½ years.

Although this method of representing a hardness index is considered a very convenient way of analyzing any series of data involving recovered asphalts from the same pavement, it should be emphasized that the softening point values of zero and 100 percent hardening will differ for each series depending on the asphalts used and their hardening characteristics.

The hardness indices were calculated for all the original and recovered asphalts in this series, as shown in table 7. Also included are the differences in the indices for the asphalts recovered immediately after mixing and after various periods of service in the pavement. The negative values shown for some of the original asphalts indicate only that the softening point was lower than 127° F., the arbitrarily chosen point of zero hardening.

The indices shown in table 7 generally reflect the same trends previously discussed for the penetration data but to a different degree. The hardening during the mixing (the hardness index of the asphalt recovered from the mix, less the hardness index of the original asphalt after refluxing and recovery) of the asphalts used in contract 1 ranged from 24 to 34 with an average of 29. For contract 2, section D–1 showed a hardening value of 32, while D–2 and D–3 were 22 and 18, respectively. There is no consistent pattern in the amount of hardening that occurred with age in the pavement. The values shown in the table for 10 and 19 years were determined by subtracting the hardness index after mixing from the hardness index of the material recovered from the pavement at the indicated age. For the 10-year period the indices ranged from 6 to 36 for contract 1 asphalts and from 4 to 22 for contract 2 asphalts. For the 19-year period, the indices for the three contract 1 sections for which data were available ranged from 26 to 52, and for the contract 2 sections, the range was 10 to 32. Since by definition these numerical values are related to the percentage of hardening at any particular stage based on the total expected hardening during the lifetime of the pavement, a wide variation in hardening characteristics is evident.

The data upon which this discussion is based represent the properties of the asphalts recovered from composite samples taken from the full width and depth of the pavement. However, it should be borne in mind that considerable variation can occur within each section. To illustrate this point, recoveries were made for samples taken from section B–1 at specific locations across the pavement after 19 years' service. The results of the tests are shown in table 8. Also included in the table are results for special tests made on material from section D–2 after 19 years. In this case, a slab of pavement was sawed in a plane parallel to the surface so as to obtain the material from the top ¼ inch, the center ¼ inch, and the bottom ¼ inch. The asphalt at each of these depths was then extracted and recovered.

Section D–2 showed the least amount of hardening based on the results of the total sample, and the difference in the penetration of the top ¼ inch and the middle ¼ inch is not significant. Both top and center samples showed more hardening than did the bottom. The hardness index value for the bottom ¼ inch is the same as that obtained for the asphalt recovered from the loose material after mixing, which indicates very little change in the asphalt at this level. The ductility values are also of interest since the material recovered from the top ¼ inch had a ductility

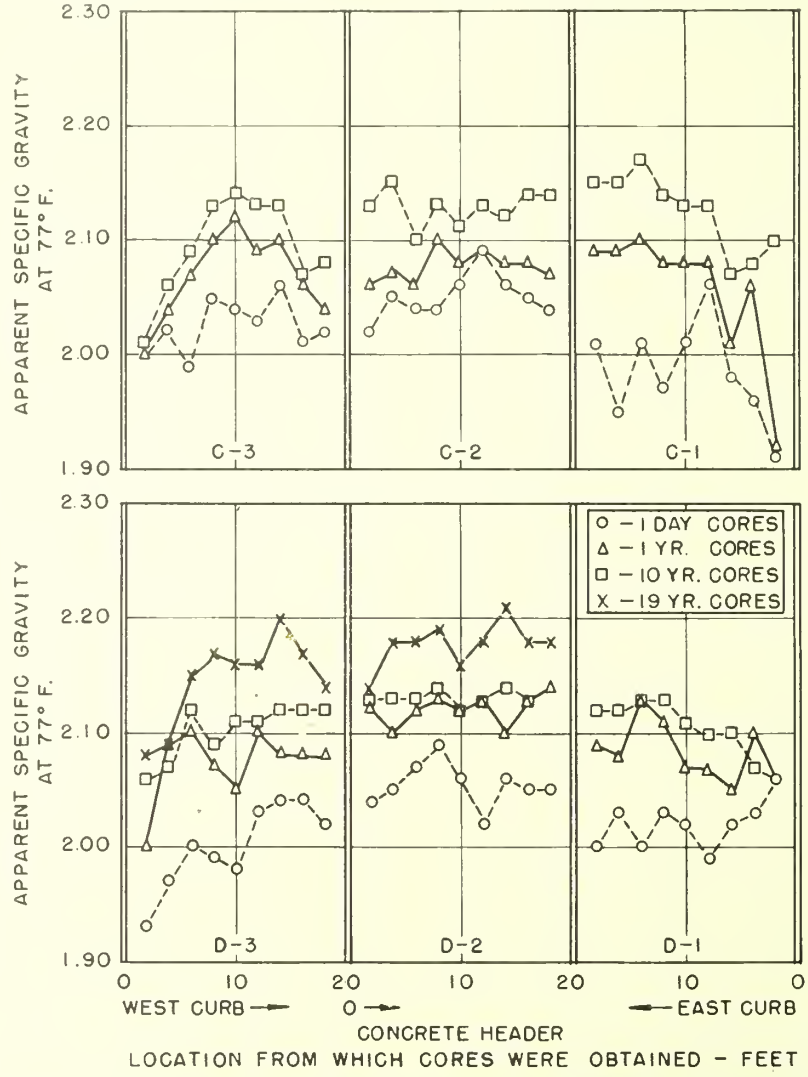

Figure 5.—Relation of density to location of samples taken from pavement test sections.

of only 25 cm. as compared with 247 cm. for the middle ¼ inch. This comparison indicates that changes other than simple hardening occur at the surface of the pavement, but unfortunately the available data are not sufficient to support definite conclusions.

Table 8.—Variations in characteristics of recovered asphalts from different locations and depths in the pavement after 19 years' service

Test section	Location: Distance from curb	Location: Portion of pavement	Bitumen (aggregate basis)	Penetration at 77° F.	Softening point	Ductility at 77° F., 5 cm./min.	Hardness index [1]
	Feet		*Percent*		*°F.*	*Cm.*	
B–1	2		12.1	12	167	3	80
	4		11.6	14	168	4	82
	6		12.2	14	165	4	78
	8		12.0	11	168	3	82
	10		11.6	9	174	3	94
	12		11.8	12	175	4	96
	14		12.2	14	168	4	82
	16		11.5	19	159	7	64
	18		12.2	23	151	15	48
D–2	--------	Top ¼ inch [2]	13.1	31	146	25	38
		Center ¼ inch [2]	12.2	32	140	247	26
		Bottom ¼ inch [2]	12.4	38	139	225	24
		Full thickness [3]	12.5	31	144	69	31

[1] See footnote 1, table 7.
[2] Samples obtained by longitudinally sawing slab; ¼ inch of material removed by saw blade for each cut.
[3] Extraction and recovery for full thickness of slab.

Table 9.—Apparent specific gravity of top inch of cores cut from pavement

Test section	Pavement age when cored, years	Apparent specific gravity of top inch of cores taken from pavement at distances from curb of— 2 feet	4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet
A–1	[1] 0	1.98	2.02	2.05	2.11	2.11	-----	2.08	2.09	2.03
	½	2.03	2.05	2.08	2.13	2.13	2.14	2.15	2.08	2.06
	1½	2.06	1.96	2.08	2.14	2.12	2.15	2.15	2.09	2.08
	2½	2.05	2.10	2.09	2.12	2.12	2.15	2.13	2.13	2.12
	4½	-----	2.04	2.09	2.12	2.15	2.18	2.17	2.13	2.15
	10½	2.04	2.07	2.08	2.12	2.14	2.14	2.15	2.18	2.14
	19½	---	------	-----	--	-- --	2.16	2.18	2.19	2.15
A–2	[1] 0	1.90	2.01	1.98	2.00	2.08	2.07	2.04	2.04	2.07
	½	1.94	2.02	2.01	2.03	2.13	2.12	2.10	2.11	2.11
	1½	1.94	2.07	2.07	2.13	2.15	2.15	2.14	2.08	2.15
	2½	1.94	2.02	2.08	2.10	2.15	2.16	2.16	2.16	2.18
	4½	------	2.13	2.17	2.15	2.17	2.16	2.16	2.20	2.16
	10½	2.02	2.07	2.12	2.15	2.20	2.17	2.20	2.19	2.15
B–1	½	1.99	2.04	2.06	2.07	2.12	2.11	2.10	2.10	2.07
	1	1.99	2.03	2.06	2.08	2.10	2.11	2.10	2.05	2.08
	2	1.95	2.02	2.07	2.06	2.11	2.07	2.09	2.08	2.10
	4	------	2.02	2.04	2.06	2.13	2.10	2.13	2.12	2.13
	10	2.00	2.04	2.06	2.08	2.12	2.12	2.13	2.14	2.16
	19	2.01	2.06	2.09	2.10	2.08	2.10	2.10	2.13	2.14
B–2	[1] 0	1.98	1.97	2.03	2.06	2.04	2.00	1.99	2.05	2.03
	1	2.02	2.01	2.05	2.07	2.07	2.06	2.07	2.05	2.07
	2	2.02	2.01	2.04	2.05	2.05	2.06	2.07	2.08	2.08
	4	------	2.02	2.01	2.06	2.05	2.06	2.07	2.08	2.08
	10	2.04	2.06	2.06	2.06	2.08	2.08	2.09	2.11	2.08
	19	--- --	2.08	2.07	2.10	2.11	2.10	2.13	2.15	2.16
C–1	[1] 0	1.91	1.96	1.98	2.06	2.01	1.97	2.01	1.95	2.01
	½	1.96	2.02	2.12	2.08	2.09	2.11	2.09	2.09	2.08
	1	1.92	2.06	2.01	2.08	2.08	2.08	2.10	2.08	2.09
	2	------	2.04	2.08	2.07	2.09	2.08	2.11	2.08	2.11
	4	------	2.05	2.09	2.09	2.08	2.13	2.11	2.12	2.12
	10	2.10	2.08	2.07	2.13	2.13	2.14	2.17	2.15	2.15
C–2	[1] 0	2.02	2.05	2.04	2.04	2.06	2.09	2.06	2.05	2.04
	½	2.08	2.11	2.10	2.10	2.10	2.09	2.11	2.10	2.11
	1	2.06	2.07	2.06	2.10	2.08	2.09	2.08	2.08	2.07
	2	2.07	2.09	2.07	2.11	2.11	2.12	2.10	2.11	2.11
	4	2.11	2.14	2.12	2.12	2.10	2.14	2.12	2.13	2.11
	10	2.13	2.15	2.10	2.13	2.11	2.13	2.12	2.14	2.14
C–3	[1] 0	---- --	2.02	1.99	2.05	2.04	2.03	2.06	2.01	2.02
	1	2.00	2.04	2.07	2.10	2.12	2.09	2.10	2.06	2.04
	2	1.98	2.06	2.08	2.11	2.10	2.09	2.11	2.08	2.08
	4	------	2.06	2.07	2.11	2.08	2.12	2.11	2.09	2.09
	10	2.01	2.06	2.08	2.13	2.14	2.13	2.13	2.07	2.08
D–1	[1] 0	2.06	2.03	2.02	1.99	2.02	2.03	2.00	2.03	2.00
	1	2.06	2.10	2.05	2.07	2.07	2.11	2.13	2.08	2.09
	2	2.04	2.07	2.06	2.08	2.08	2.08	2.10	2.08	2.08
	4	------	2.07	2.08	2.09	2.10	2.11	2.12	2.11	2.11
	10	2.06	2.07	2.10	2.10	2.11	2.13	2.13	2.12	2.12
D–2	[1] 0	2.04	2.05	2.07	2.09	2.06	2.02	2.06	2.05	2.05
	1	2.09	2.08	2.13	2.13	2.10	2.08	2.12	2.08	2.12
	2	2.08	2.08	2.12	2.14	2.11	2.10	2.13	2.10	2.12
	4	2.12	2.10	2.12	2.13	2.12	2.13	2.10	2.13	2.14
	10	2.13	2.13	2.13	2.14	2.12	2.13	2.14	2.13	2.14
	19	2.14	2.18	2.18	2.19	2.16	2.18	2.21	2.18	2.18
D–3	[1] 0	1.93	1.97	2.00	1.99	1.98	2.03	2.04	2.04	2.02
	1	2.00	2.09	2.10	2.07	2.05	2.10	2.08	2.08	2.08
	2	2.05	2.06	2.08	2.08	2.08	2.09	2.11	2.08	2.09
	4	------	2.08	2.10	2.10	2.08	2.10	2.10	2.09	2.10
	10	2.06	2.07	2.12	2.09	2.11	2.11	2.12	2.12	2.12
	19	2.08	2.09	2.15	2.17	2.16	2.16	2.20	2.17	2.14

[1] After final rolling.

Changes in Physical Characteristics of Pavement

The densities (expressed in terms of apparent specific gravity) of the top inch of specimens cut from the sheet asphalt surfacing throughout the 19-year period of service are given in table 9. The results for all periods up to and including 10 years are for cores cut from the pavement, whereas those shown for the 19-year period are for rectangular specimens cut from the pavement. The results show the effect of the transverse location of the sample in each traffic lane on the degree of compaction and stability. Although the specific gravities reported in table 9 are for the top inch of the specimen, values for the full thickness of the pavement showed no significant differences.

The general trends are illustrated in figure 5 in which specific gravity values for sections C–1, C–2, and C–3 from contract 1, and sections D–1, D–2, and D–3 from contract 2 are plotted with respect to their relative position in the pavement. The density of the pavement nearest either curb is generally the lowest with a general trend toward higher density in the middle of each segment of the pavement. There is a corresponding decrease in density in the proximity of the concrete headers, but the difference is not as pronounced as that caused by the curbs. The greatest density gains in all sections occurred in the first 12 or 18 months of service. The density patterns after rolling and after consolidation by the traffic follow normal expectations. Generally, the effect of traffic was to decrease the irregularities in the specific gravity results, but the general pattern of higher density in the center of each section was retained. For sections C–3 and A–2 the differences in the densities between the center and edges of the pavement showed a tendency to increase, but for all other sections the densities became more uniform. The initial specific gravities for section D–1 did not follow the general trend since those nearest the east curb were generally high, but after traffic compaction this trend was essentially eliminated.

The Hubbard-Field stability test results reported in table 10 generally show a consistent increase in values up to the end of about the 4-year period, while the results at the end of 10 years are very inconsistent. Much of this variation is believed to be caused by the inherent difficulties in obtaining satisfactory cores from pavements and the test limitations when the asphalt has hardened to the extent shown after 10 years. In such cases it is extremely difficult to eliminate crushed edges or the effects of minute cracks which may form during the sampling. For this reason, cores for the stability tests were not taken at the end of the 19-year period.

Comparison of Laboratory Compaction With Field Compaction

From the standpoint of design practices, the relation between the minimum voids obtained in the actual pavement and the voids obtained by laboratory design procedures is

Table 10.—Hubbard-Field stability of top inch of cores cut from pavement

Test section	Pavement age when cored, years	Hubbard-Field stability (in pounds) at 140° F. of top inch of cores cut from pavement at distances from curb of—								
		2 feet	4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet
A–1	0[1]	------	1,110	1,190	1,500	1,570	------	1,330	1,470	1,080
	½	1,410	1,580	2,150	2,150	2,200	2,370	2,300	2,300	2,080
	1	1,890	1,450	2,380	2,700	2,450	2,700	2,100	2,150	2,450
	2	------	2,180	2,280	2,550	2,680	2,400	2,630	2,630	2,720
	4	------	2,120	2,960	3,780	4,620	3,880	3,860	3,880	3,950
	10	3,460	3,970	------	4,140	3,270	------	2,800	4,020	------
A–2	0[1]	------	1,130	810	920	1,540	1,300	1,130	1,140	1,080
	½	500	1,410	1,550	1,710	2,620	2,140	1,970	2,280	2,060
	1	------	1,800	2,000	2,600	2,950	2,200	2,450	2,750	2,100
	2	------	1,800	2,300	2,380	2,600	2,680	2,380	2,430	2,500
	4	------	1,700	4,720	4,790	4,470	4,300	4,360	3,250	4,070
	10	2,560	------	3,040	2,860	------	2,840	2,640	------	3,130
B–1	0[1]	975	1,175	1,350	1,650	2,050	1,750	2,050	1,950	1,650
	1	1,550	1,550	2,050	2,050	2,500	1,950	2,100	1,850	2,050
	2	1,160	1,500	2,160	2,000	2,100	2,250	1,920	2,360	2,000
	4	------	2,150	2,450	3,130	3,600	3,420	3,550	2,450	2,650
	10	2,580	2,860	------	3,240	3,160	------	2,860	2,780	------
B–2	0[1]	1,110	1,200	1,520	1,860	1,420	1,240	1,180	1,400	1,350
	1	1,870	1,700	2,050	2,100	2,250	2,000	2,100	2,100	2,000
	2	1,500	1,620	1,680	1,950	2,330	2,150	2,280	2,420	2,450
	4	------	2,500	2,280	2,900	2,500	3,000	3,560	3,280	3,360
	10	2,850	3,260	------	2,960	3,400	------	2,940	3,060	------
C–1	0[1]	680	620	940	1,400	1,120	860	1,020	720	940
	½	1,050	1,300	2,350	2,200	2,250	1,850	2,150	2,400	2,050
	1	1,150	2,200	1,900	2,100	2,200	2,450	2,150	2,350	2,250
	2	------	2,300	2,370	2,570	2,430	1,950	2,450	2,000	2,150
	4	------	2,500	4,050	3,600	3,100	4,250	1,800	4,300	4,200
	10	2,800	2,840	------	3,080	3,050	------	2,640	2,450	------
C–2	0[1]	950	1,060	1,250	1,160	1,400	1,710	1,400	1,220	1,040
	½	1,850	2,300	2,150	1,800	2,100	1,800	2,000	2,550	1,900
	1	2,400	2,250	2,200	2,150	2,200	2,150	2,450	2,100	2,350
	2	1,900	2,600	2,200	2,550	2,550	2,300	2,350	2,450	2,400
	4	2,220	3,650	3,620	3,500	3,050	3,660	2,980	3,780	3,750
	10	2,730	2,150	------	2,500	2,760	------	2,740	2,510	------
C–3	0[1]	------	1,290	1,200	1,740	1,370	1,600	1,510	1,130	1,100
	1	1,550	2,100	2,300	2,700	2,600	2,800	2,400	2,650	2,550
	2	1,600	2,250	2,900	2,700	2,650	2,500	2,600	2,750	2,450
	4	------	3,250	3,200	4,300	3,200	3,850	3,900	4,100	3,400
	10	3,300	3,760	------	6 050	4,000	------	3,380	4,100	------
D–1	0[1]	1,340	1,030	1,200	1,010	1,100	1,280	990	1,220	1,160
	1	2,050	2,250	2,050	1,900	2,050	2,450	2,250	2,050	2,050
	2	1,700	1,900	1,950	2,150	2,100	2,250	2,150	2,000	2,100
	4	------	2,600	3,420	3,630	3,770	2,750	3,370	4,080	------
	10	2,740	2,800	------	2,860	2,970	------	2,680	2,840	------
D–2	0[1]	1,450	1,560	1,720	1,960	1,680	1,390	1,520	1,250	1,110
	1	2,150	------	2,200	2,250	2,250	2,400	1,900	2,250	1,900
	2	2,400	2,250	2,600	2,350	2,500	2,350	2,600	2,200	2,250
	4	4,400	4,510	------	------	3,950	4,750	4,500	3,900	3,760
	10	3,820	3,320	------	2,640	3,070	------	4,160	2,840	------
D–3	0[1]	650	900	1,090	1,180	900	1,210	1,550	1,470	1,260
	1	1,300	1,650	2,150	1,450	1,800	1,750	1,650	1,650	1,700
	2	1,600	1,750	2,100	2,070	2,250	2,220	2,450	2,020	1,800
	4	------	3,300	3,600	3,400	3,256	3,200	2,560	3,700	3,450
	10	2,740	2,940	------	4,100	3,480	------	3,780	3,130	------

[1] After final rolling.

of interest. The actual voids are affected by the amount of compaction resulting from both the roller and traffic. The amount of compaction is also affected by the position of a particular sample in the pavement. It is believed that the maximum density obtained at any particular time should most nearly reflect the influence of the mix design. The relation of the voids calculated at this maximum density over a period of years to those calculated from laboratory specimens is shown in table 11. The table also includes the mixture voids calculated by determining the void content of the aggregate by the vibratory method as described by Pauls and Goode (*5*). The voids in the mixture were then calculated by reducing these aggregate voids by an amount equivalent to the volume of asphalt used. The percentage of voids obtained by laboratory compaction of the plant mixture from each section at 3,000 pounds per square inch, using the double plunger method, is also shown. The latter method is essentially the same as the ASTM method (*6*).

In general, both laboratory methods gave lower voids than the minimum obtained immediately after rolling. The 3,000-pound molding by the double plunger gave lower voids in most instances than those calculated from the vibratory compaction of the aggregate. The average for all sections after rolling was 8.9 percent. This compares with the averages of 7.9 percent for the vibratory method and 7.4 percent for the double plunger method and a load of 3,000 pounds per square inch. After 1 year of traffic, the minimum voids in the pavement approached those obtained by the two laboratory methods of compaction, the latter ranging from 6.1 to 9.6 percent and averaging 7.5 percent for all sections.

Because of the many variables involved, the calculated voids shown in table 11 do not show a definite trend. In some sections, for example A–1, there was very little change in the minimum voids (maximum specific gravity) after the 4-year period; in others a general trend toward lower voids was noted as in the case of sample C–2; while in still others a more or less stable pattern was evident between 4 and 10 years, but there was a considerable drop in the minimum voids after 10 years, for example section D–2. This section had minimum void values of 5.7 for the 2-, 4-, and 10-year periods but dropped to 2.6 at 19 years.

Surface Abrasion Loss

One factor that affects the results of both the physical tests on the pavement and the characteristics of the recovered asphalt is the amount of abrasion loss caused by the traffic on the pavement. It is difficult to obtain an accurate measure of this wear because of the changes in the density of the pavement with time. However, as indicated by the data in table 9, density changes for later years were small for most sections. Accordingly, thickness measurements were not made until 1940, and it is believed that the greater portion of any measured change since that date can be attributed to the wear of the pavement rather than consolidation. The thickness measurements obtained are shown in table 12.

Although complete data were not obtained after the 19-year period, the trends noted are

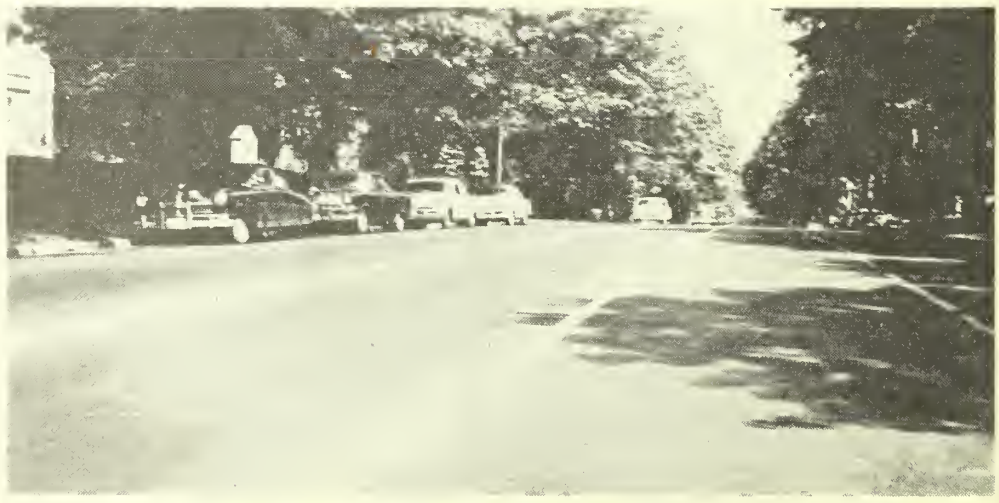

Figure 6.—General view of test area (sections D–1, D–2, and D–3) in April 1955.

significant. Generally it is indicated that the wear for most sections was accelerated in the period between 10 and 19 years. This can be attributed to two causes: (a) increased hardening of the asphalt, and (b) a considerable increase in the amount of traffic for the later years. This increase in traffic is illustrated in table 13. A second trend is the greater loss of thickness for portions of sections which probably carried the most traffic. The center lanes that would normally carry less traffic showed less wear. A third possible significant item is the relation of the amount of wear to the hardness of the recovered asphalt. Sections that showed the larger amounts of hardening also showed the greatest wear. For example, sections B–1 and B–2 in which the asphalts were reduced to penetrations of 14 and 13, respectively, decreased in average thickness from 1.47 inches to 0.96 inch (a 35-percent loss) for B–1 and from 1.53 to 1.09 inches (29-percent loss) for B–2. Section D–2 which had the highest penetration of the recovered asphalt after 19 years showed a 10-percent loss (0.18 inch) in thickness, from 1.62 to 1.46 inches.

The fact that the abrasion loss in heavy traffic can be such a large proportion of the surface course introduces an additional difficulty in evaluating the amount of hardening that occurs in the asphalt. It has been established that the asphalt in the top portion of the pavement hardens more rapidly than that below the surface; therefore it is the harder material that is always being lost. The recovered asphalt must necessarily be obtained from the pavement that remains, which obviously will be softer than would be the case if all the abraided material could be retained.

Table 11.—Minimum void content of pavement samples compared with calculated voids of laboratory compacted specimens

Test section	Maximum theoretical density [1]	Percentage of voids in pavement samples						Percentage of voids in laboratory compacted specimens	
		After rolling	After 1 year	After 2 years	After 4 years	After 10 years	After 19 years	By vibratory test	By 3,000 lbs./sq. in. double plunger
A–1	2.28	7.5	-------	-------	4.5	4.3	4.3	7.7	7.0
A–2	2.28	8.7	-------	-------	3.5	3.5	-------	7.4	5.6
B–1	2.27	[2] 6.6	7.0	7.0	6.1	4.8	5.7	6.8	7.3
B–2	2.29	10.0	9.6	9.1	9.1	7.8	5.6	11.0	9.1
C–1	2.27	9.2	7.4	7.0	6.1	4.4	-------	7.0	6.7
C–2	2.27	7.9	7.4	6.6	5.7	5.2	-------	6.8	6.4
C–3	2.29	10.1	7.4	7.8	7.4	6.5	-------	8.3	8.1
D–1	2.29	10.0	6.9	8.2	7.3	6.9	-------	8.2	8.6
D–2	2.27	7.9	6.1	5.7	5.7	5.7	2.6	5.9	7.3
D–3	2.29	10.9	8.2	7.9	8.2	7.3	3.9	9.5	7.8
Average	--------	8.9	7.5	7.4	6.4	5.6	4.4	7.9	7.4

[1] Based on specific gravities of asphalt and mineral aggregates.
[2] Results of cores sampled at age of 6 months.

Table 12.—Measured thicknesses of pavement surfaces at various times

Test section	Age of pavement, years	Thickness (inches) of sheet asphalt at distances from curb of—								Average thickness, inches
		4 feet	6 feet	8 feet	10 feet	12 feet	14 feet	16 feet	18 feet	
A–1	4½	1.20	1.45	1.60	1.80	1.65	1.60	1.57	1.47	[1] 1.57
	10½	1.19	1.25	1.50	1.75	1.62	1.62	1.57	1.37	[1] 1.54
	19½	------	------	------	------	1.25	1.00	1.13	1.00	[1] 1.10
A–2	4½	1.50	1.45	1.55	1.50	1.40	1.47	1.30	1.05	1.40
	10½	1.34	1.31	1.28	1.31	1.31	1.33	1.16	.98	1.24
B–1	4	1.55	1.60	1.60	1.60	1.32	1.35	1.32	1.40	1.47
	10	1.55	1.62	1.50	1.25	1.19	1.25	1.25	1.06	1.33
	19	1.38	1.25	1.06	.81	.62	.81	1.00	.75	.96
B–2	4	1.80	1.65	1.50	1.35	1.45	1.45	1.55	1.47	1.53
	10	1.75	1.40	1.50	1.50	1.19	1.19	1.37	1.31	1.40
	19	1.38	1.25	1.25	.81	.88	1.00	1.13	1.00	1.09
C–1	4	1.30	1.40	1.50	1.30	1.25	1.15	1.10	1.10	1.26
	10	1.28	1.44	1.38	1.34	1.00	1.00	1.00	.91	1.17
C–2 [2]	4	1.50	1.40	1.35	1.30	1.20	1.30	1.50	1.55	1.39
	10	1.38	1.31	1.13	1.25	1.28	1.25	1.50	1.50	1.33
D–1	4	1.55	1.60	1.70	1.50	1.65	1.55	1.57	1.45	1.57
	10	1.56	1.62	1.68	1.75	1.75	1.68	1.50	1.38	1.62
	19	[3] 1.98	1.75	1.56	1.44	1.06	1.13	1.25	1.31	1.36
D–2 [2]	4	1.62	1.70	1.70	1.65	1.75	1.65	1.60	1.30	1.62
	10	1.68	1.75	1.50	1.75	1.75	1.69	1.50	1.18	1.60
	19	1.50	1.50	1.50	1.75	1.56	1.50	1.44	1.00	1.46
D–3	4	1.25	1.40	1.40	1.35	1.50	1.45	1.32	1.30	1.37
	10	1.22	1.44	1.44	1.31	1.22	1.22	1.18	1.22	1.28
	19	1.06	1.13	1.19	.94	1.00	.94	1.00	.88	1.02

[1] Average for 12- to 18-foot distances from curb only.
[2] Center section distances are measured from west header.
[3] Not included in average.

Table 13.—Traffic volumes in the vicinity of test sections

Location of control station	Direction of traffic	Total vehicles, 24-hour period		Test sections affected
		1946	1956	
South of Tilden Street	Northbound	10,878	33,200	A–1, B–1
Do	Southbound	10,470	32,100	A–2, B–2
South of Albemarle Street	Northbound	11,381	28,200	C–1, C–2
Do	Southbound	10,974	33,300	C–2, C–3
South of Military Road	Northbound	6,634	23,400	D–1, D–2
Do	Southbound	6,080	26,400	D–2, D–3

Visual Inspection of Pavement

In general, the behavior of the pavement as a whole would be considered satisfactory although there was some indication of progressive deterioration. Visual inspection was made at the time of each sampling but no definite trends were noted. Figure 6 illustrates the general appearance of the test area in April 1955 when final samples were taken. Spalling and cracking of the sheet asphalt surfacing at some of the larger reflection cracks occurred relatively early and became more pronounced in later years as the asphalt hardened and the portland cement concrete base adjoining the joint progressively deteriorated. Figure 7 is an example of one of the more advanced cases of this deterioration. There was no tendency of the surfacing to displace or shove under traffic except at bus stops. Six bus stop areas were rebuilt in recent years with cement concrete pavement.

Sheet asphalt was unsatisfactory in the gutter areas. Figure 8 shows a typical condition after 10 years. This is in contrast to the good condition of the paving brick gutters used on other portions of the two contracts.

Moisture was observed on the surface of the cement concrete base and, to a lesser degree, in the binder course and lower levels of the sheet asphalt surfacing course when samples were taken. There was some indication of slight stripping in the binder but none in the sheet asphalt surfacing. It is believed that the stripping was insufficient to affect the behavior of the pavement except in the gutter area.

The occurrence of moisture between the cement concrete base and the bituminous surfacing after resurfacing old concrete is well known and is a matter of some concern. There is some uncertainty as to the manner by which this water accumulates, but it is believed to

Figure 7.—View of section B–2 in April 1955, showing cracks at joint and over areas of base failure.

Figure 8.—View of sheet asphalt surface failure in June 1946 (south of section C–1) caused by surface water at the curb line.

be similar to the phenomenon that often occurs with impervious paint films on houses when moisture diffuses through the walls from the inside and collects under the film, resulting in blistering and peeling. In the case of the pavement, the water vapor moves upward through the sub-base and through the concrete base, either directly or by capillary flow through the pores of the concrete during periods when the surface is warmer than the base. Because this vapor cannot readily escape through the surface, it remains trapped and will condense as moisture in the binder course or at the interface of the concrete and the asphaltic material.

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Airphoto Analysis of Terrain for Highway Location Studies in Maine

By ERNEST G. STOECKELER, Airphoto Interpretation Specialist, University of Maine, and WILLIAM R. GORRILL, Soils Engineer, Maine State Highway Commission [1]

Airphoto interpretation techniques are especially useful for highway engineering terrain studies in wilderness areas where little or no detailed information on geology or soils is available. In Maine, time-consuming and expensive field reconnaissance surveys were reduced to a minimum by the intelligent use of aerial photography. Detailed field investigation and laboratory testing are still required to obtain information for final design purposes, especially in critical areas.

The Maine State Highway Commission has successfully employed airphoto interpretation techniques for obtaining a variety of information valuable in various phases of highway engineering. The four types of strip studies described in this article are only a few of many possible applications of this field. It is highly probable that more intensive and specialized photo interpretation studies will be made in the near future in Maine as well as throughout the nation.

THE Airphoto Interpretation Project at the University of Maine is financed jointly by the Maine State Highway Commission and the Bureau of Public Roads, U. S. Department of Commerce. The function of this project is to provide information on soils, drainage, and other natural and cultural features which might be useful in all phases of highway engineering from preliminary planning to construction operations.

Engineering soils, drainage, and materials maps have been prepared for an area of approximately 3,500 square miles. These maps were made by quadrangles, an area of approximately 200 square miles, at a scale of 2 inches equal 1 mile. Precedence was given to the areas where most construction was anticipated. These maps are especially useful in the preliminary planning phases of location studies and in the preparation of cost estimates.

In addition to these maps, a number of strip studies, using airphoto interpretation techniques, were accomplished. The main subject of this article is a discussion of various types of photo-strip studies and their applications in highway engineering.

Types of Strip Studies

Approximately 300 linear miles of new location and relocation strip studies were completed during 1957. Areas covered by these projects are shown in figure 1. Individual projects varied in length from 3 miles to over 100 miles. In practically all cases, aerial photography was performed specifically for the State Highway Commission. Photographic scales ranged from 1:5,400 to 1:20,000, depending on the type of study desired. Most of the photography was undertaken in the spring after snow had disappeared and before new foliage was developed. Photographs taken in the fall after deciduous foliage has been shed are also acceptable. The seasonal aspect is of special significance in Maine, because at least 90 percent of the areas studied to date were in densely wooded wilderness sites where terrain interpretation from summer photography is difficult because surficial features are often masked by the forest canopy.

These studies covered proposed routes of the Interstate highway system as well as a number of highway relocations. The type of data and methods of presentation varied somewhat for individual projects. The four major types of strip studies included (1) engineering soils, (2) drainage, (3) gravel haul, and (4) highway relocation and reconstruction.

The four types of strip studies were made in the reconnaissance phase as well as in the preliminary survey. Based on information furnished by these strip studies alternate routes were compared, cost estimates were prepared, and in some cases preliminary designs were made.

Engineering Soils Strip

The Maine reconnaissance engineering soils classification system was devised by the authors specifically for engineers. While this system is based on a geological approach, the nomenclature is given in engineering terminology. This system reduces the number of soil units to a minimum, yet provides the information required by local highway engineers. The following eight map units are tailored for Maine terrain and climatic conditions, and are adaptable to airphoto interpretation techniques which require a minimum amount of field checking.

Rock (*R*).—Ledge within 5 feet of the surface.

Boulder Granular (*BG*).—Granular drift.

Boulder (*B*).—Boulder clay drift.

Granular (*G*).—Sand or gravel.

Fines (*F*).—Silt or clay.

Swamp (*S*).—Surficial peat less than 3 feet thick.

Peat (*P*).—Surficial peat more than 3 feet thick.

Water (*W*).—Generally more than 2 feet deep.

Figures 2–4 are stereograms that illustrate these soil units.

For preliminary planning soils studies, 1:20,000 scale photography was used. In most cases a single 3-mile-wide strip centered on a preliminary line was provided for delineating soils areas. Occasionally two or three parallel strips were flown where more widely separated alternate routes were to be studied. The minimum size area delineated was approximately 5 acres, except for such elongated forms as esker ridges and narrow valley swamps.

In addition to soils delineations, annotations were made on the photographs emphasizing significant terrain features such as deep bogs, extensive rock outcrops, steep slopes, and other obstacles which should be avoided. Annotations depicting favorable terrain conditions were also indicated.

Figure 5 (p. 148) is an uncontrolled mosaic showing a 5-mile segment of the proposed 110-mile section of the Interstate highway system between Orono and Houlton, Maine. The soils information was transferred directly

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

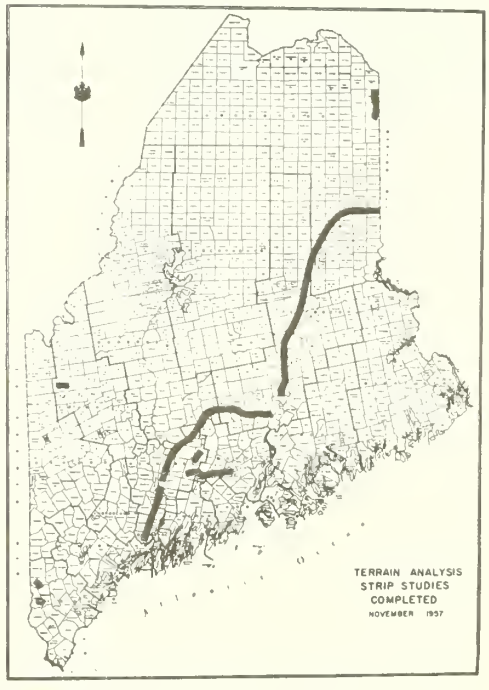

Figure 1.—Airphoto strip studies completed in 1957.

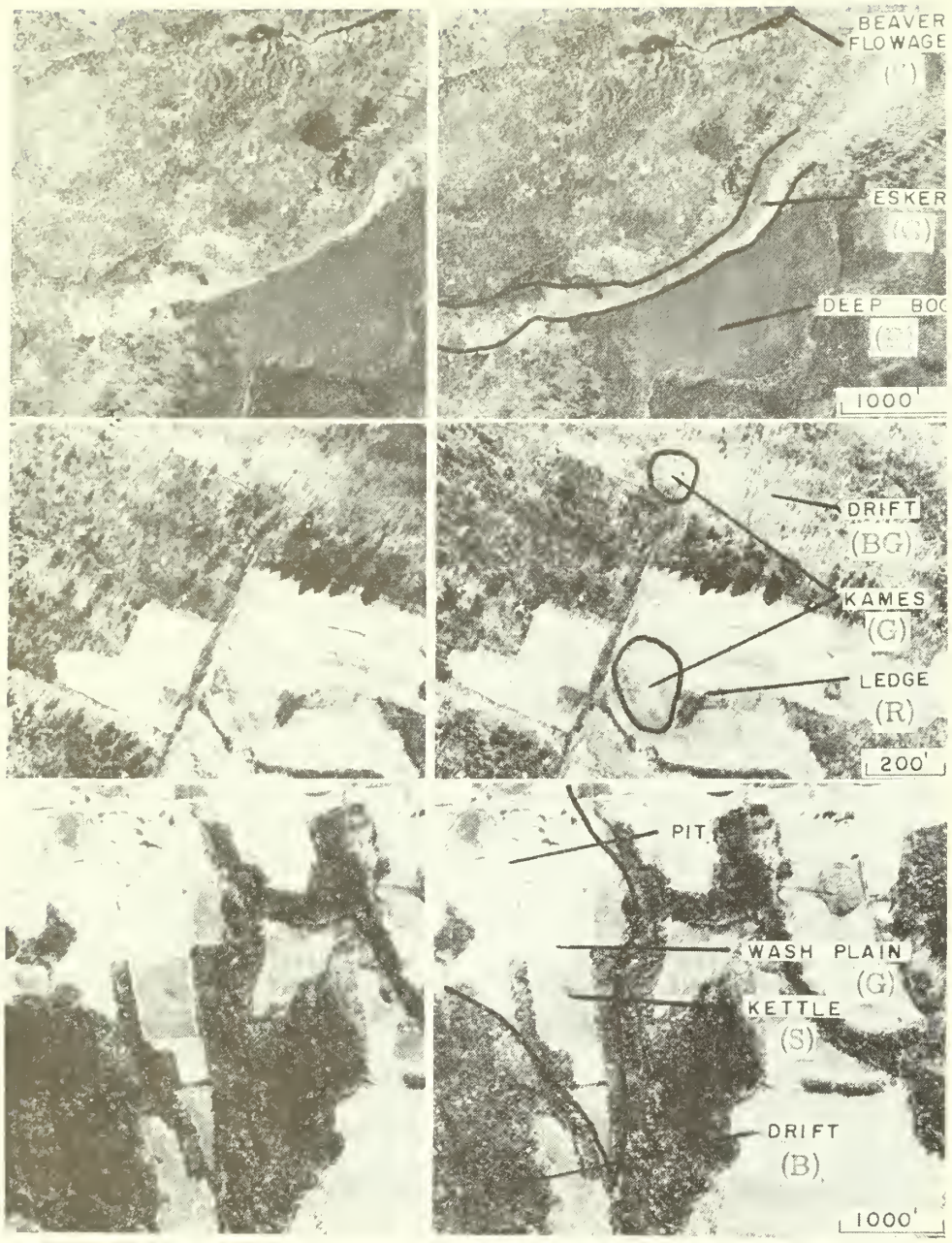

Figure 2.—Stereograms of several glacial landforms. (Courtesy of the Maine State Highway Commission.)

from the airphotos to a profile which had been prepared by photogrammetric methods. The P-line and stationing were placed on the photographs used for soils interpretation to facilitate the procedure of transferring soils data to the profile sheet as shown in figure 6.

With the engineer and the interpreter working together, the grade line was often adjusted to take advantage of terrain characteristics. For example, other conditions permitting, cuts were frequently increased at locations where ledge probably would not be encountered, especially where good granular excavation was anticipated. In areas where rock outcrops were numerous, the grade line was adjusted to minimize the amount of rock excavation. In addition to adjusting the grade line, the preliminary horizontal alinement was modified to avoid difficult terrain and also to take advantage of favorable construction areas. This information was used for the preparation of a preliminary cost estimate of the Interstate highway system.

Another more detailed engineering soils strip study was made of a 30-mile portion of the Interstate highway system in a more advanced planning stage. Airphotos for this study were taken in the spring at a scale of 1:5,400. A private photogrammetric concern was engaged to prepare topographic maps of a 3,200-foot-wide strip at a scale of 1 inch equals 200 feet and a contour interval of 5 feet. For this project the soils information was transferred from the photographs to the topographic sheet, thus enabling the location engineer to evaluate the engineering and economic aspects of a number of possible road lines within the 3,200-foot-wide strip. After detailed analyses of a number of possible lines were made, a centerline was selected and photogrammetric methods were used to determine cross sections.

With large scale photography of this type the interpreter can delineate small problem areas such as rock outcrops and swamps of less than one-tenth acre. In boulder-strewn glacial deposits, and even in wooded areas, it is often possible to measure individual boulders for the purpose of estimating the quantity of ledge-size boulders (larger than 1 cubic yard) contained in the deposit. Areas where seepage, settlement, and frost action might occur were also annotated on the photographs.

The detailed engineering soils map and annotated photographs were then provided to field geologists for detailed soils investigation. Suggested probe and drill-hole locations were pinpointed on the photographs to assist the geologist in determining actual soil type boundaries. Additional explorations were made at the discretion of the geologist in order to obtain an adequate soils profile. After an evaluation was made of the field investigation and laboratory analysis of the samples, the soils map was modified and extended to include detailed information on critical areas. Frequently the field work was expedited by trails and access roads indicated on the photographs. By following this procedure, a maximum amount of soils and terrain data was obtained with the minimum expenditure of manpower, time, and money.

Drainage Strip Studies

In Maine, as elsewhere, the design of drainage structures in relocation projects is often based on observations of the past performance of existing structures not far distant from the proposed new location. If the existing structure proved adequate over a long period of years a comparable design for the nearby location is used. However, for most of the proposed Interstate highway system routes and many major highway relocations, this method of design cannot be employed because often no existing nearby structures are available for evaluation. The U. S. Geological Survey topographic sheets having a scale of 1 inch equals 1 mile and a contour interval of 20 feet are often inadequate for determining watershed areas, especially small watersheds of less than 1,000 acres. In addition, many quadrangle sheets in Maine were compiled over 20 years ago and some before World War I. Because of these inadequacies, highway engineers have recently exhibited considerable interest in photohydrological studies.

The suitability of photography for drainage interpretation studies is dependent principally on scale and season of photography. In densely wooded areas the seasonal aspect is probably of prime importance. In summer photography, many drainageways and minor relief breaks are hidden by the forest canopy. In these cases the interpretation of vegetation provides the only clue to the existence of a drainageway. Due to climatic conditions in Maine, photographs taken during the spring break-up period just after the snow has disappeared are excellent for drainage interpretation purposes. During this period every little drainageway and depression is filled with water, and the photographic patterns are very easy to interpret. Even in dense forests con-

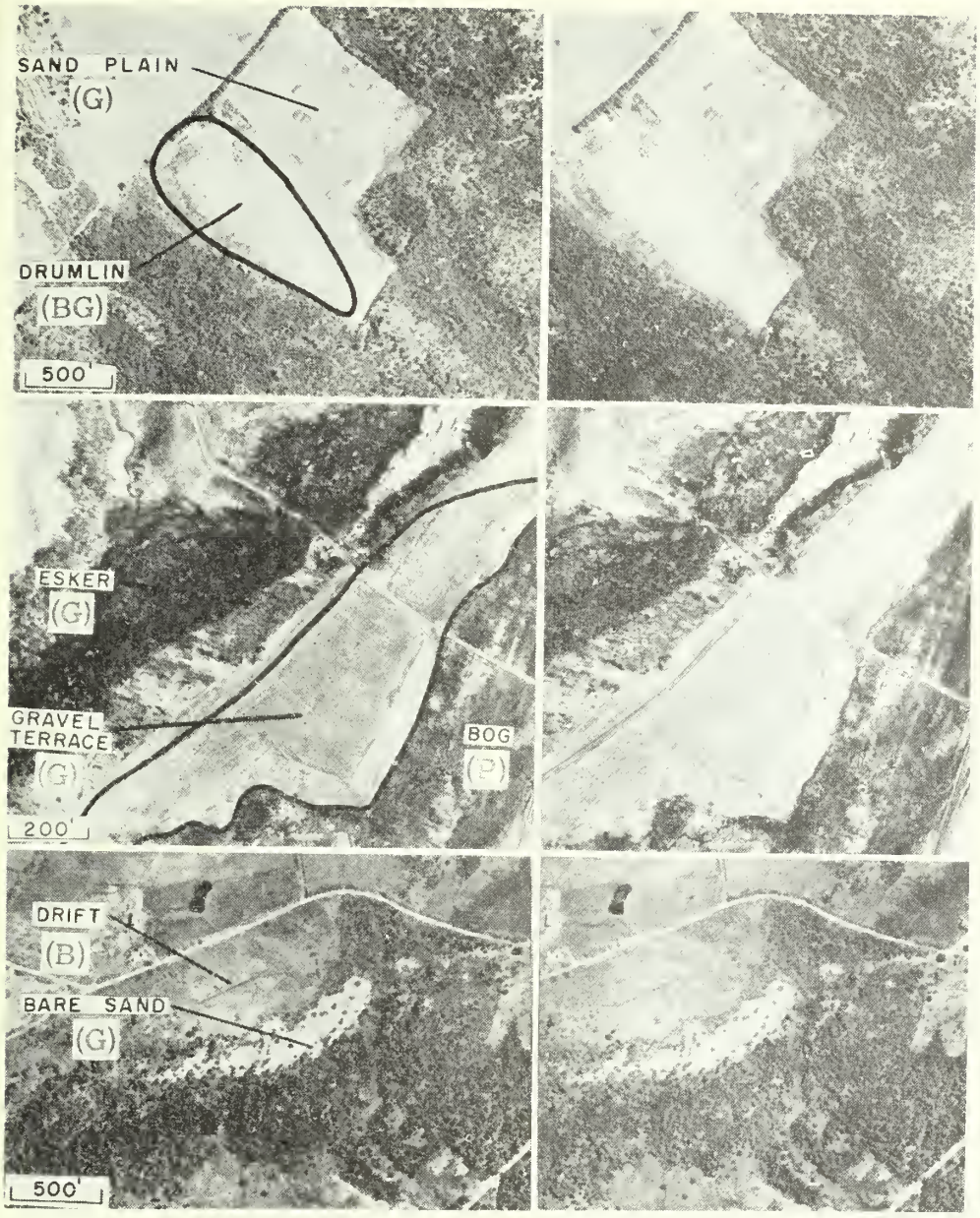

Figure 3.—Stereograms of glacial, water-lain and wind-blown deposits. (Upper and lower panels, courtesy of the Sewall Company, Old Town, Maine; middle panels, courtesy of the Maine State Highway Commission.)

taining 75 percent coniferous trees, there are enough canopy breaks formed by the leafless deciduous trees to permit the interpreter to see a sufficient amount of the earth's surface so that even small drainageways can be traced.

For preliminary reconnaissance drainage surveys, photography having a scale of 1:20,000 is adequate. Figure 7 is an uncontrolled mosaic of a drainage study made along a portion of a proposed Interstate highway system route. Many smaller drainageways and minor ridge lines delineated on the original 1:20,000 photographs were omitted in this illustration for reproduction purposes. Drainage strip studies of this type were made on approximately 240 linear miles of the Interstate highway system in Maine.

Area, slope, and cover-type information for each individual watershed was provided to the highway engineer who calculated the preliminary drainage structure sizes. This information was then transferred to the profile using the same procedure described previously (see fig. 6). In addition, information on nearby drainage structures, the existence of water storage areas in individual watersheds, possible seepage areas, and similar hydrological data were noted on the profile sheet. This information was used for the preparation of preliminary cost estimates.

For more detailed drainage studies, photographs having a scale of 1:5,000 to 1:10,000 are preferred. In densely wooded areas, especially on relatively flat terrain, small drainageways are difficult to identify on 1:20,000 photography. Ridge lines can also be plotted more accurately on large scale photography.

Gravel Haul Strip

Another type of airphoto strip study made in connection with the location of the Interstate highway system was an analysis of the availability and accessibility of granular materials along a 110-mile section. Much of the proposed route was located in a wilderness area several miles from existing roads. The purpose of the study was to provide a basis for estimating the amount of overhaul which could be anticipated at various locations on this large project. In Maine, the maximum free-haul distance is 2 miles.

An engineering soils strip study of this area was completed previously. Granular deposits delineated on these photographs were rapidly restudied to obtain the overall view of the entire length of the 110-mile section. The section was then subdivided into 8 projects to facilitate study of shorter sections for analysis purposes. Arbitrary separation points between projects were based on an evaluation of a number of factors including (1) major stream crossings, (2) number, spacing, and location of potential gravel deposits in the particular area, (3) location of towns and major highway crossings where entrance lanes would probably be required, and (4) general characteristics of the terrain.

Figure 8 (p. 150) is an uncontrolled mosaic of a 5-mile section showing the preliminary line stationing, and possible access routes from the delineated deposit to the project. Figure 9, a stereogram detail of a portion of figure 8, illustrates the type of information which is presented in the gravel haul report.

As seen in table 1, the haul distance was indicated by the following road types:

Improved 2-lane road.—Paved or all-weather gravel roads permitting two-way traffic; no improvements required.

Improved 1-lane road.—Narrow, gravel-surfaced roads, such as country lanes and main haul logging roads which may have turnouts at irregular intervals, likely found in a very soft and inoperable condition during the spring break-up period; improvements to varying degrees should be expected.

Unimproved road.—Single lane roads, either dirt or with a thin gravel surfacing; considerable improvements even for one-way traffic are definitely required.

Road required (new road).—Access road from deposit to project is nonexistent or the need for a more direct route is indicated. For construction through wooded areas the letter *W* is appended to the distance, and the letter *C* is added if the area is cleared or brush covered. Where streams must be crossed, the width of the stream in feet is tabulated.

Volume estimates shown in table 1 are for areas designated on the original photography. Where long continuous eskers were found near the preliminary line, select 2,000- to 3,000-foot portions of the deposit were outlined and the volume was appended with a plus sign, which means that additional materials could be obtained by extending the indicated boundaries. In cases where the distance between deposits was rather long, a number of access roads to the project were plotted, especially if a large amount of material was available. In plotting proposed access routes, downhill hauls were chosen and unfavorable terrain was avoided where

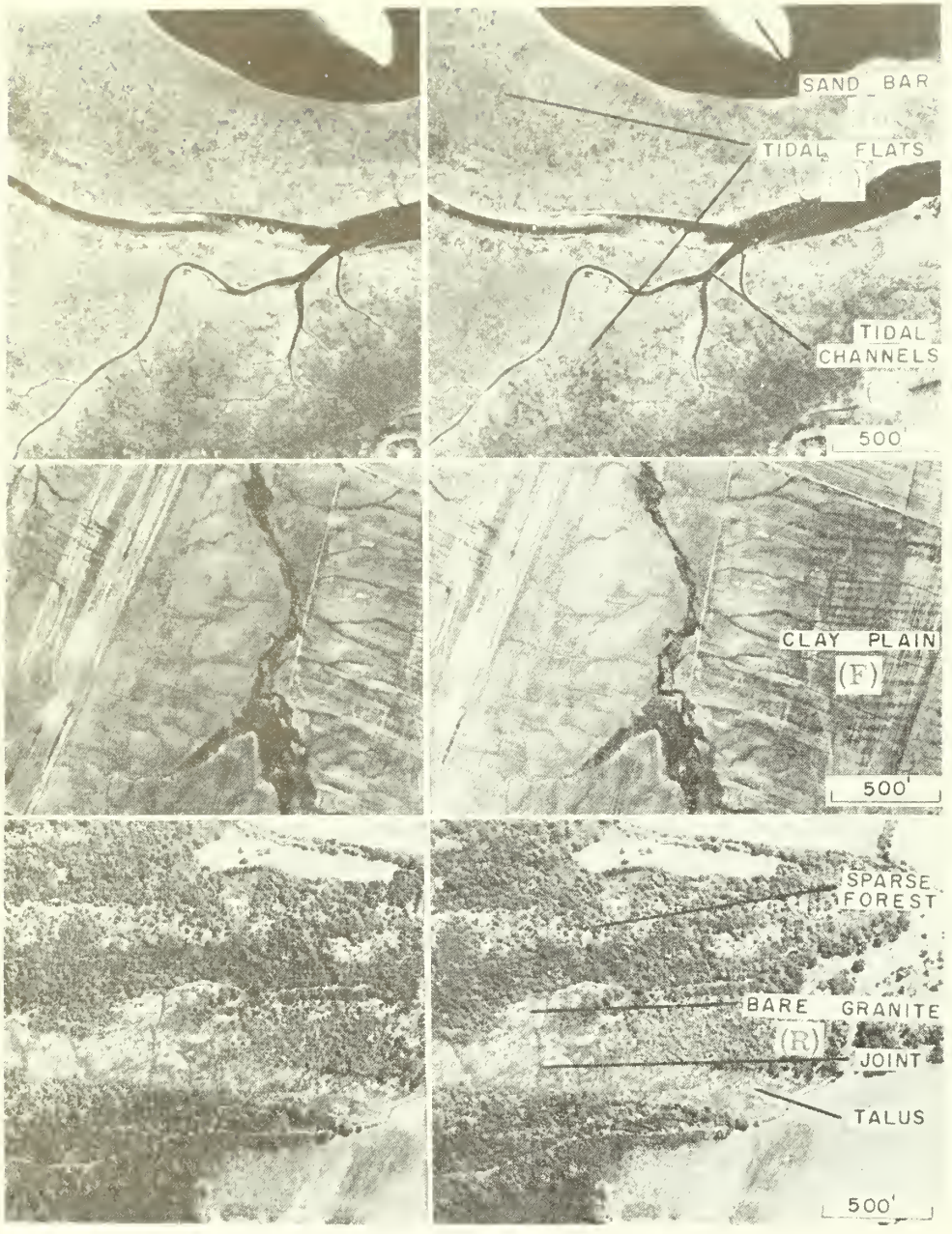

Figure 4.—Stereograms of fine-grained marine and lacustrine deposits and ledge terrain. (Upper and lower panels, courtesy of Maine Cooperative Wildlife Research Unit; middle panels, courtesy of the Maine State Highway Commission.)

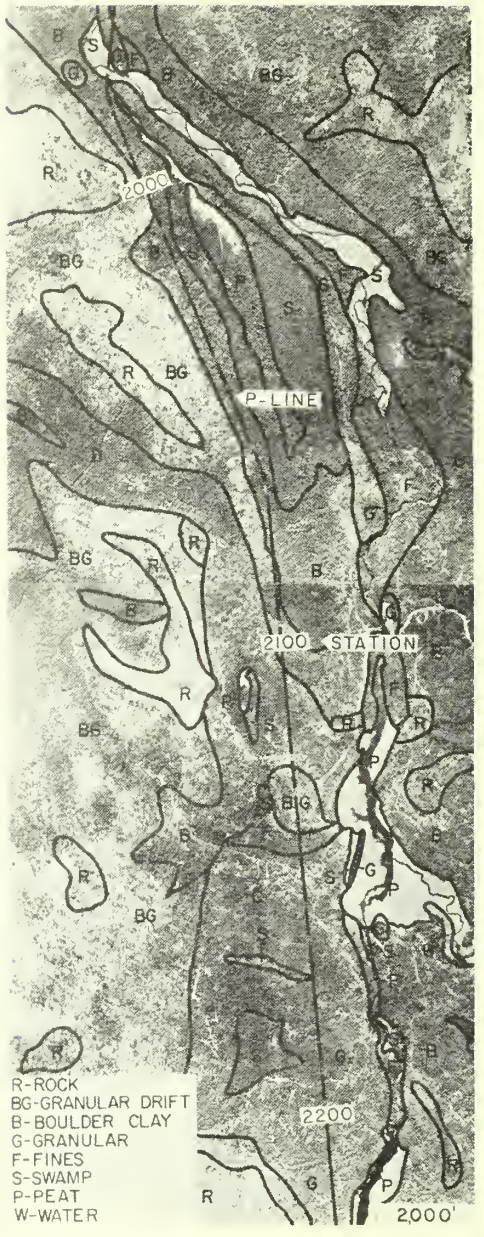

Figure 5.—Uncontrolled mosaic showing engineering soils areas (courtesy of the Maine State Highway Commission); stations were placed at 1,000-foot intervals on the originals.

possible. Existing roads were utilized where feasible. In figures 8 and 9, note that suggested routes cross the narrowest portions of the extensive swampy area found between the deposits and the preliminary line.

A brief description of each project was also included in the gravel haul report. The type of information presented is illustrated by the following quotation from the actual report pertaining to the area shown in figure 8.

"Project 4.—Canadian Pacific R. R.–Medway (Station 2047 to 2813). There are no public roads of any type in this 14.5-mile section except in the immediate vicinity of Medway. A 1-lane main haul logging road which is operable about 11 months per year crosses the preliminary line at several points. In the area immediately north of the railroad there are numerous secondary logging roads and sled trails which are generally too winding to be of much value as access roads. A logging camp, located at Station 2103, 2,800 feet left of centerline, is the only building within a mile of the preliminary line in the 25-mile section between Howland and Medway. The most promising development possibility appears to be Deposit 26 located at the midpoint of the project. Because of the probable dearth of granular materials between Stations 2400 and 2800, a number of access roads are indicated from Deposits 26 and 28 to various points on the project. In several instances it would be necessary to span the East Branch of Medunkeunk Stream which is about 30 feet wide. Most feasible crossings are at Stations 2516, 2618, and 2635. At other points it would be necessary to traverse swampy terrain varying in width from 250 to 1,000 feet. Deposit 29 is . . . ."

Approximately 90 deposits located in the vicinity of the centerline of this 110-mile section of the Interstate highway system were described in the gravel haul report. An analysis of this information indicated that less than 20 percent of the total mileage would probably involve overhaul charges. This information was incorporated in preliminary cost estimates.

In addition to the gravel haul analysis, a number of gravel prospect spot studies were completed during 1957. These studies, not shown in figure 1, usually covered an area of approximately 100 square miles centered on a construction project located in a quadrangle where materials maps have not been prepared to date. Any available photography having scales ranging from 1:3,600 to 1:70,000 was used for interpretation. Considerable photographic coverage of Maine is already in the possession of the State Highway Commission and the University of Maine Photo Interpretation Project. Figures 10 and 11 are

reductions of photographs having original scales of 1:70,000 and 1:40,000, respectively, showing extensive gravel deposits on single photographs covering vast areas. It is estimated that over 30-million cubic yards of granular material are in the area shown in figure 10, which covers approximately 80 square miles. Very small scale photographs are useful for reconnaissance studies of extensive areas. Multiple coverage at scales of 1:20,000 and 1:5,000 is ideal for interpretation purposes.

Potential granular deposits were delineated on the photographs, and suggested spots for field checks were pinpointed. Access to potential deposits was plotted on the photographs. In areas devoid of easily recognizable cultural features, conspicuous natural features

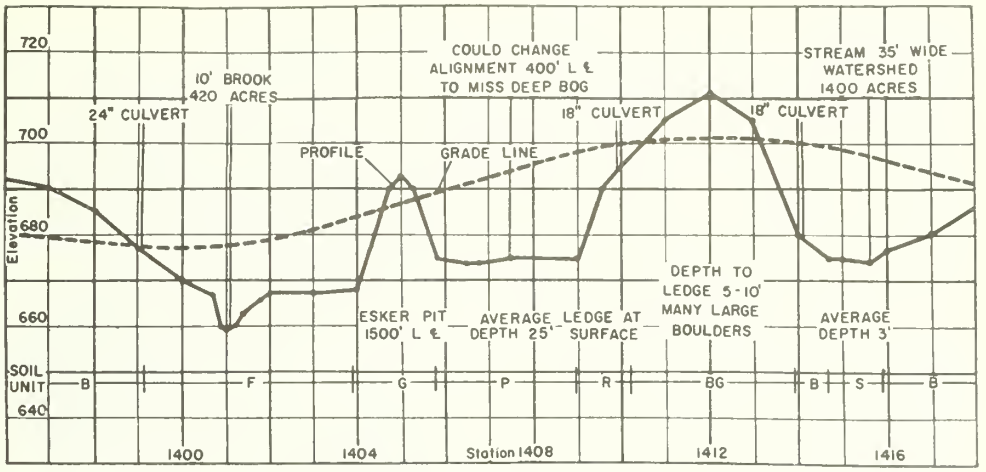

Figure 6.—Profile with engineering soils and drainage annotations.

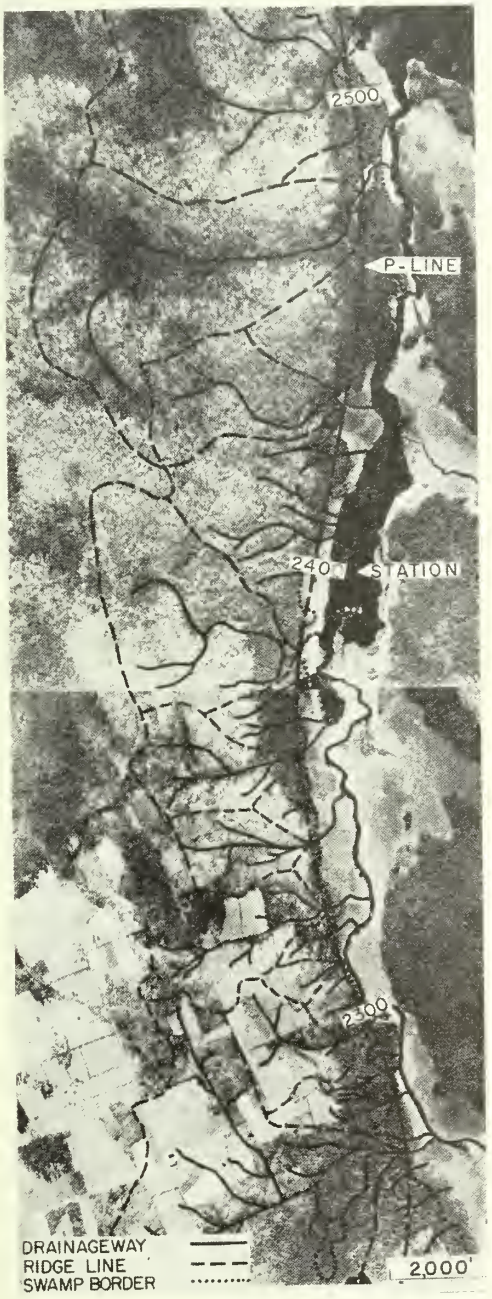

Figure 7.—Uncontrolled mosaic of drainage delineations (courtesy of the Maine State Highway Commission); watershed areas were obtained directly from the photographs.

such as rock outcrops, local forest clearings, and creeks were annotated on the photographs. Distances and compass bearings were also indicated for the convenience of the field checkers. Identification numbers were assigned to each potential deposit on both the photograph and a U. S. Geological Survey topographic sheet.

The annotated photographs and maps were furnished to the field geologists who determined actual field conditions. Volume estimates were prepared and samples were taken for laboratory tests. This information was made available to bidders.

Relocation and Reconstruction Strips

Terrain analyses were made of six proposed relocation projects scheduled by the State Highway Secondary Division. These strip studies varied in length from 3 to 25 miles. Several of the longer strips were flown for the Highway Commission at a scale of 1:12,000. For the shorter studies the best available photography was used.

These studies required close cooperation between the secondary highway engineer and the interpreter. Because of many years' experience in observing the behavior of a particular stretch of road, the engineer was intimately acquainted with practically every bad curve, dip, frost boil, and excessive grade for the entire length of the proposed construction project. He was also well informed on future planning which might influence relocation, traffic counts, local needs and similar important information which cannot be gleaned from the photographs by the interpreter. Consequently, the first stage of relocation studies consisted of the engineer's review of the entire length of the project with the interpreter. The engineer indicated general segments of the road where relocation or improvements of one type or another were anticipated. These locations were noted on the photographs by the interpreter. Pertinent information directly or indirectly affecting the location of the new road was noted directly on the photographs; for example, "feeder road will be relocated in 5 years so the PC cannot be north of this point," or "centerline should not be more than 1,000 feet east of this intersection."

After the interpreter was thoroughly briefed by the engineer, soils areas were delineated in a band of varying width which would amply cover any possible relocation line. Conspicuous natural features such as swamps and rock outcrops were annotated. Significant cultural features such as houses, stores, cemeteries, schools, churches, gas stations, feeder roads, and trails were traced directly from the photograph, using the central portion of the photograph to minimize distortion. Strip tracings were made in 4- to 6-mile sections, or 20 to 30 inches per drawing at a scale of 1:12,000. After the tracing of features for the entire strip was completed, the project was subdivided into shorter sections to facilitate analysis. Depending on terrain and road characteristics, the subsections varied in length from 2,000 to 15,000 feet.

Table 1.—Gravel haul data sheet

Deposit number	Volume, cu. yd.	Access at station number	Haul distance (miles)				
			Improved 2-lane road	Improved 1-lane road	Unimproved road	New road [1]	Total
26	300,000+	2435	-------	0.5	------	-------	0.5
26A	300,000+	2466	-------	1.2	0.6	-------	1.8
26B	300,000+	2495	-------	1.9	.8	-------	2.7
26C	300,000+	2539	-------	1.9	.9	0.1 W	2.9
27	200,000+	2516	-------	-------	-------	[2] 7 W	.7
28	200,000+	2618	-------	-------	1.9	[2] 2 W	2.1
28A	200,000+	2635	-------	-------	2.2	[2] 1 W	2.3
28B	200,000+	2682	-------	-------	3.3	.1 W	3.4
28C	200,000+	2702	-------	-------	3.7	.1 W	3.8

[1] W indicates wooded land. [2] 30-foot bridges required.

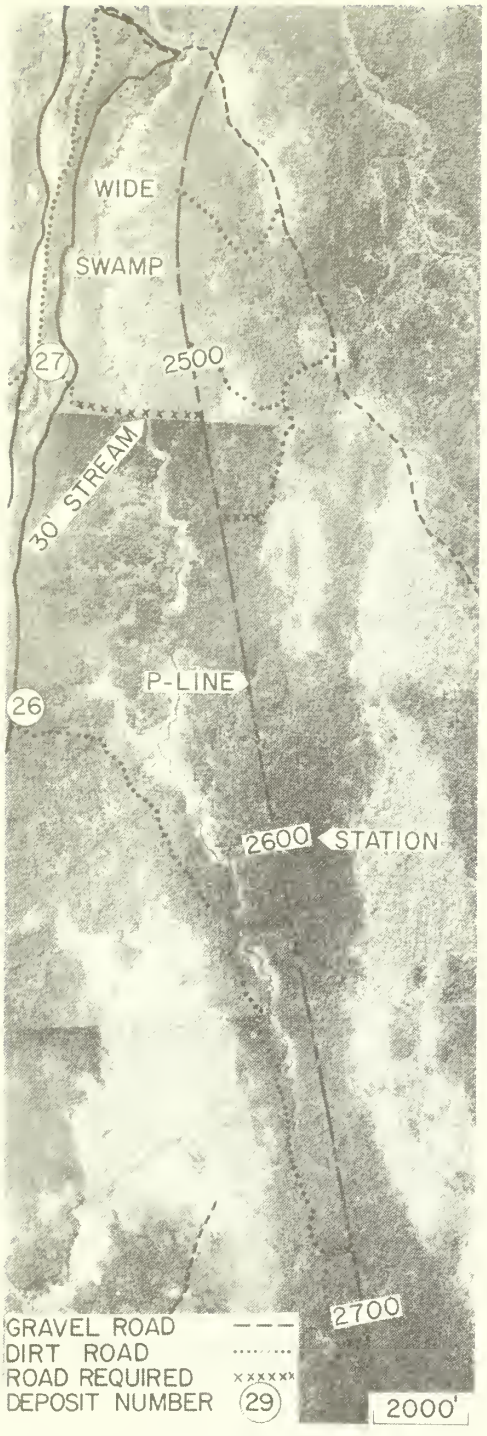

Figure 8.—Uncontrolled mosaic of a gravel haul study. (Courtesy of the Maine State Highway Commission.)

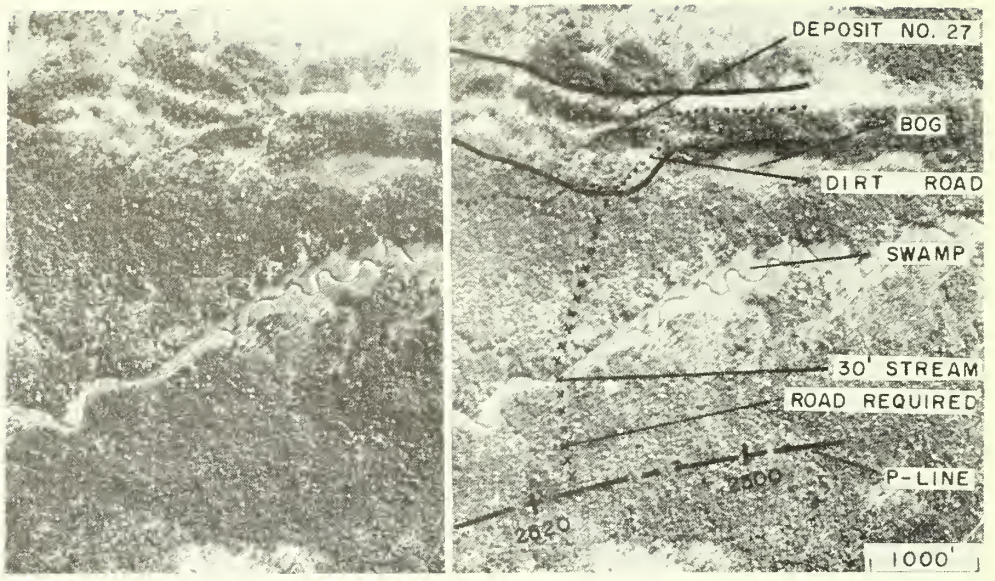

Figure 9.—Stereogram of a portion of the area shown in figure 8. (Courtesy of the Maine State Highway Commission.)

Figure 10.—Esker system in the Penobscot Valley, Maine. (Courtesy of the U. S. Army Map Service.)

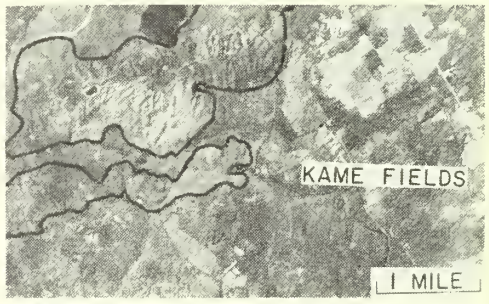

Figure 11.—Vertical photograph of extensive kame fields in western Maine. (Courtesy of U. S. Geological Survey.)

The photographs were then restudied in detail to analyze natural and cultural features, and an evaluation of each subsection was made. Instead of plotting a suggested preliminary line and perhaps one or two alternates, a recommended band for detailed study was outlined on the tracing. The intent of this approach was to establish boundaries for a strip which offered the least construction difficulties by avoiding unfavorable terrain where possible. The width of the recommended band ranged from 50 to over 500 feet, depending principally on the length of the cutoff, terrain conditions, and the location of cultural features. This allowed the location engineer considerable leeway in plotting various grade-alinement design combinations to meet the road requirements, still keeping within the recommended band. Alternate bands were given if feasible. In instances where the improvement amounted to a simple road widening, a single line was indicated in

(Continued on next page)

lieu of a band. Spot elevations at major topographic breaks within the recommended band were pinpointed on the tracing. Locations of existing gravel pits and also potential deposits were annotated. The engineer was provided with ozalid prints, transparent overlays, and overlapping airphotos.

A brief verbal description of each subsection was included in relocation study reports. To illustrate, the following is quoted from a report describing the subsection shown in figure 12. "Two recommended bands for detail study are indicated at the Palermo cutoff. The first, labeled B–1, consists of a series of three widenings and one 2,500-foot cutoff located 300 feet south of the main village. This band traverses two granular soils areas which may yield suitable borrow. Band B–2, a wide sweeping curve located 1,000 feet south of the village, would require 8,000 feet of new construction, of which 400 feet is through a shallow swamp and about 3,000 feet is over ledge terrain. The southern edge of the band skirts a very deep bog and the northern boundary is limited by a cemetery. Based on terrain considerations only, the northern portion of Band B–2 is preferred. A wash plain containing an operable pit is located less than 2 miles from the west end of the project . . ."

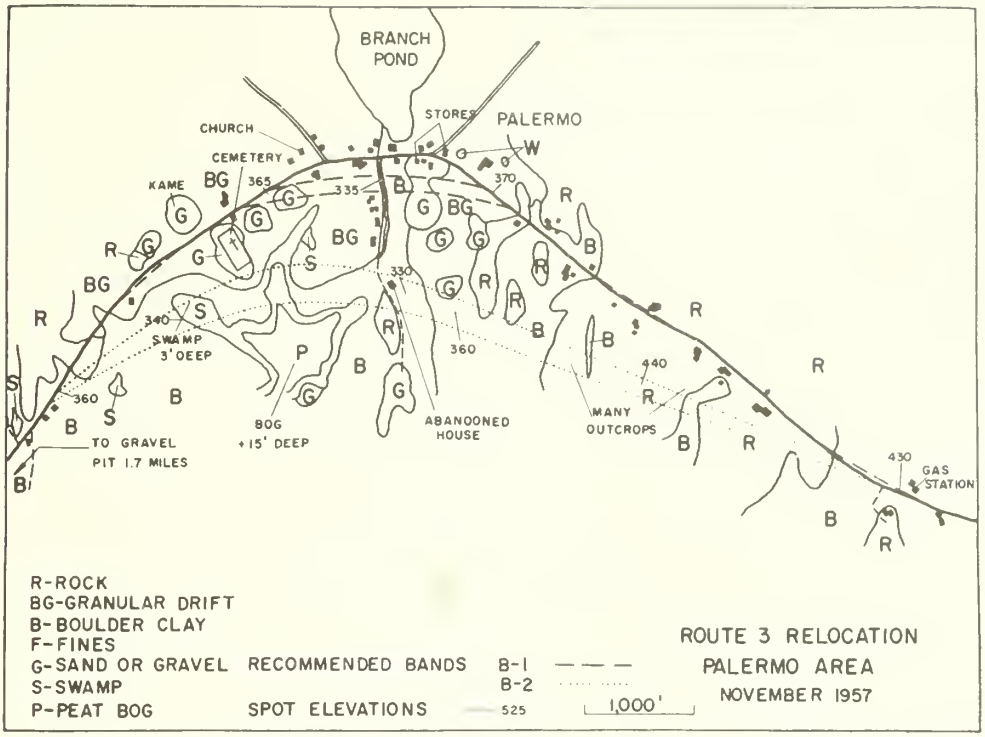

Figure 12.—Secondary highway relocation study showing recommended bands for detailed study.

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VOL. 30, NO. 7 APRIL 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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PUBLISHED BIMONTHLY BY THE BUREAU OF PUBLIC ROADS, U.S. DEPARTMENT OF COMMERCE, WASHINGTON

In this issue: A study of the impact of suburban development upon residents' travel to the central business district of Washington, D.C.
(Pennsylvania Avenue, Washington, D.C.)

IN THIS ISSUE

Trends in Travel to the Central Business District by Residents of the Washington, D.C., Metropolitan Area, 1948 and 1955 153

New Publications 176

Lost Mixing Time of Dual Drum Pavers: a motion picture 176

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 7 *April 195*

C. M. Billingsley, *Editor*

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Washington 25, D.C.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

Trends in Travel to the Central Business District by Residents of the Washington, D.C., Metropolitan Area, 1948 and 1955

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported [1] by JACOB SILVER,
Transportation Economist

This article evaluates the changes that took place between 1948 and 1955 in the relationship of the central business district to the metropolitan area. Many of the findings presented here, which are based on quantitative data, confirm the unsupported views of persons concerned with the changing urban structure.

Although the number of trips destined to the central business district increased during the 7-year period, the proportion of total area trips to the central area declined. Suburbanization, accompanied with development of commercial establishments, has had a definite impact upon the business center of the city. Although the central business district remained a major terminus for residents trips, it was less so in 1955 than in 1948.

CENTRAL BUSINESS DISTRICTS in today's metropolitan areas are taking on new meaning in the urban structure. In the past 2 decades the urban population has increased at a very rapid rate, with a general tendency on the part of families to live farther from the central city. With this expansion has come a development of new outlying commercial and service facilities which, with an increased number of new employment centers outside the central business district (CBD), are now carrying on functions formerly associated with the central area.

What has been the impact of the growth of these functions upon the CBD and the travel to it? The need for information to answer this question has become increasingly important in order that planners and legislators may have a better understanding of the changing urban relationships in this very complex section of our cities. From the data obtained from the 1948 and 1955 home-interview travel surveys [2] which were sponsored by the Regional Highway Planning Committee for metropolitan Washington, D.C., in cooperation with the Bureau of Public Roads, an attempt has been made to measure this impact as it relates to travel.

Summary of Findings

The most important travel characteristic reflecting the change in the relative economic character of the CBD between 1948 and 1955 was the decreasing proportion of trips to the CBD in relation to total travel. The pattern in which the percentage of intra-area trips destined to the CBD decreased progressively with distance and varied with socioeconomic conditions and historical trends in a lateral direction around the CBD remained similar in both years. In 1955, however, residents of each of the geographical subdivisions studied made a smaller proportion of their intra-area trips to the CBD than in 1948. Although the CBD remained a land-use complex of major trip attraction, it was less an attractor of trips in 1955 than in 1948.

Of those residents who did make trips to the CBD, the more important changes in the geographical pattern of their distribution took place with distance from the central area rather than with direction. Although there were fluctuations in the directional distribution of CBD trips, the pattern in 1955 remained approximately the same as in 1948. The pattern of distribution as related to distance indicated a more significant shift, however, as an increasing percentage of the CBD trips were made by individuals residing at greater distances from the central area. These changes followed a pattern related to the distribution of population and residential growth, and also to the increasing uniformity in the frequency of trips per dwelling unit.

As indicated by the number of trips for each purpose, the functions carried on within the central area were changing. The retail segment of the CBD became relatively less important as the number of trips for nonretail activity increased and those for shopping decreased.

This does not imply that shopping trips to the central area will decline until they become insignificant. The volume of shopping trips should increase if greater accessibility to the central area is provided and shopper convenience is improved. Nevertheless, by providing the same attracting forces outside the CBD that previously caused residents to make trips to the CBD, a greater choice in direction of movement has been given area residents.

Of those residents who were attracted to the central area for one purpose or another, the most notable change was the increasing use of the private automobile. The general increase in automobile ownership permitted a greater use of this mode of travel for all purposes for which trips would be attracted to the CBD. As automobile ownership increases in those subdivisions that were among the lowest in 1955, and as the number of trips to the CBD by individuals residing at greater distances from the CBD where transit service is less convenient increases, the use of the private automobile in traveling to the CBD should increase even more.

In Washington, D.C., as in other cities, many CBD functions have expanded into the fringe area surrounding the CBD because of increasing costs of land, rent, taxes, etc. As a result, many of the travel characteristics peculiar to the central area have become apparent in the fringe area. To obtain a more comprehensive picture of travel to the CBD, this area should be analyzed in future studies.

Scope of Study

The metropolitan area of Washington, D.C., included in the 1948 transportation study is shown in figure 1. To develop travel data to the central area and to determine the relative effects of distance as well as direction upon a person's travel to the CBD, the study area was subdivided into rings and sectors. Sector divisions of the 1948 survey were used to indicate direction and 2-mile rings, as measured from the Zero Milestone,[3] were used to show distance. Together they make up what henceforth will be called ring-sectors, and all

[1] This article was presented at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959.

[2] The procedures used in these studies are described in the *Manual of Procedures for Home Interview Traffic Study*, which is available by purchase from the Public Administration Service, 1313 East 60th Street, Chicago, Ill.

[3] Although Zero Milestone was not the exact center of the CBD, it was chosen for use in this study because it had been employed in an earlier study *Travel to Commercial Centers of the Washington Metropolitan Area*, by Gordon B. Sharpe, Highway Research Board Bulletin 79, 1953.

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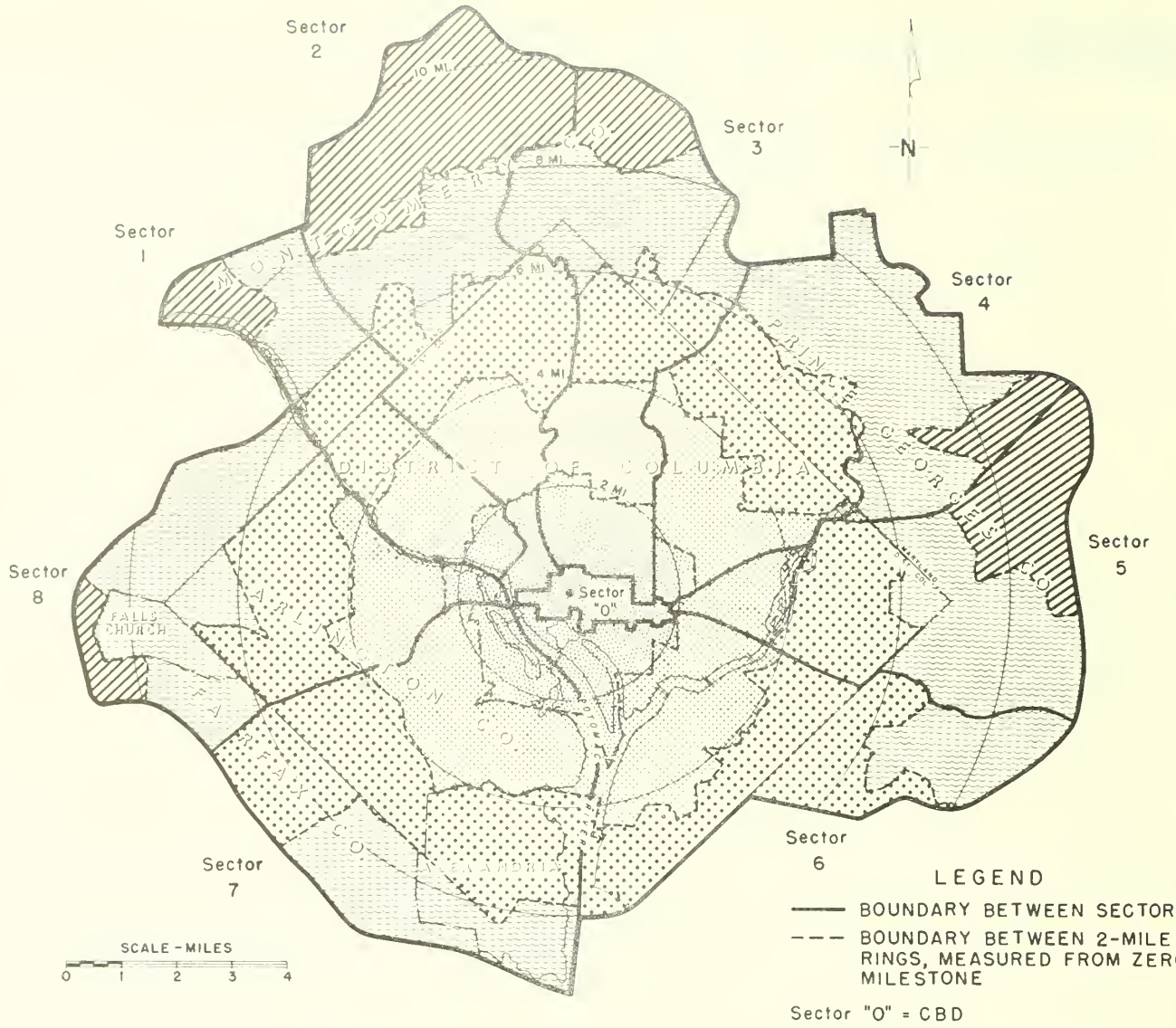


Figure 1.—Area of 1948 Washington, D.C., metropolitan transportation study, subdivided by ring-sectors.

trip data were grouped into these subdivisions. Ring-sectors are comprised of a number of transportation zones grouped according to the distance from the CBD of each zone's 1948 centroid of population. Because of the location of the centroids of population and the irregular shapes of transportation zones, the stippled areas in figure 1 tend to overlap the distance rings.

The area of the repeat study in 1955 was extended beyond the 1948 limits because of the growth in residential development. To have comparable data for a trend analysis, however, only those trips by persons residing within the 1948 area were used. This procedure did not bias the 1955 data to any great extent since the 1948 area included residents who made 92 percent of the trips within the 1955 cordon and 95 percent of the residents trips destined to the CBD.

The Washington CBD, as defined in this analysis, is shown in figure 2. Trips to this area, included in the study, were average weekday trips related to the traveler's zone of residence, even though the trips may have originated elsewhere. Although the trip zone of origin would provide a truer picture of direction and distance of actual travel to the CBD, experience in sampling has shown that place of residence and distribution of population are more stable and reliable factors when determining travel characteristics, both for the purpose of estimating future travel patterns to the CBD and for comparing travel patterns in two or more cities.

The trip purposes studied were limited to three groups: work, shop, and other. The category of "other" included business, medical-dental, school, social-recreation, eat meal, change mode of travel, serve passenger, and home trips.

There were several reasons for this grouping of trip purposes: first, for purposes of analysis, it was simpler; second, trips to work and to shop are the purposes most closely associated with trips to the CBD; and third, residents making trips for each of the reasons grouped in the "other" category made less than 20 percent of such trips to the CBD on an average weekday.

The modes of travel analyzed included automobile driver, automobile passenger, taxi passenger, and mass-transit passenger. Trips by taxi and truck operators in the course of their daily work and all pedestrian travel were excluded. Pedestrian trips, however, must be kept in mind in any discussion of data concerning trips coming from within the first 2-mile ring.

Characteristics of Study Area

Though there were changes within the Washington CBD, such as buildings demolished, parking facilities created, new office buildings erected, etc., the basic physical outline did not change greatly between 1948 and 1955, and it continued to have the commercial and employment functions associated with this complex type of land use. In addition, the CBD also houses many Federal Government activities, which are a major source of employment for area residents.

The number of persons employed in the CBD remained almost constant, increasing less than 2 percent between 1948 and 1955 (189,100 to 192,800 persons employed).[4]

[4] Data concerning employment, income, and retail sales were obtained from selected statistics prepared by the mass transportation staff of the National Capital Planning Commission and the National Capital Regional Planning Council.

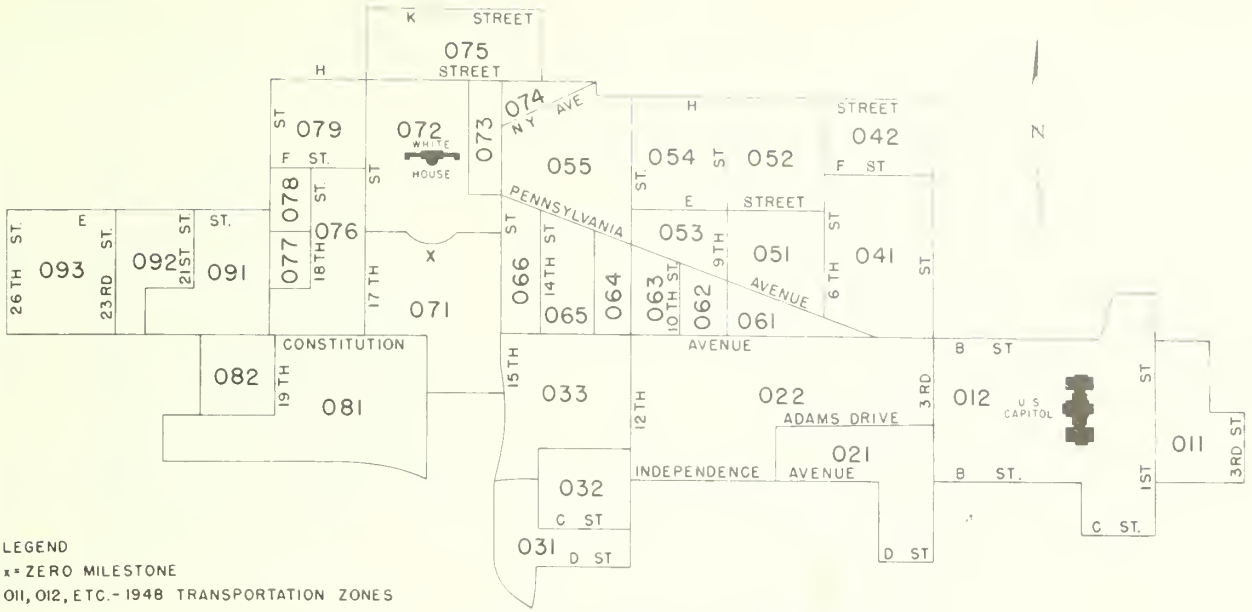


Figure 2.—Central business district of Washington, D.C.

Employment opportunities, however, had increased outside the CBD at a greater rate than within. The proportion within the CBD decreased from 31 to 27 percent, or from a ratio of 1 job for every 6 persons to 1 for every 8 persons living in the area.

Retail sales within the CBD remained approximately the same, about $367 million for each of the 2 years. This is not a true comparison, however, when considering the changed value of the dollar. It was found that prices in 1955 had increased generally by 11.4 percent over 1948. When adjusted by the Consumer Price Index for metropolitan Washington as published by the Bureau of Labor Statistics, 1955 retail sales amounted to $325 million, or a relative loss of 11 percent, as measured in 1948 dollars. Moreover, even though retail sales for the metropolitan area increased, the proportion of these sales made within the CBD, on a constant dollar basis, decreased from 26 to 19 percent.

Since the close of World War II, population growth in the Washington, D.C., area has followed the general trend of most large metropolitan areas in the United States. Between 1948 and 1955, population within the area, as determined by the travel studies, increased 31 percent, from 1,110,000 to 1,454,000 persons (table 1). Approximately 95 percent of this growth occurred beyond 4 miles of the CBD.

A comparison of the percentage distribution of population and dwelling units between 1948 and 1955 reveals that the more significant relative changes occurred between the 2-mile distance rings rather than between sectors around the CBD. The relative distribution of population had changed between rings by as much as 6 percentage points from what it had been in 1948, while 7 of the 9 sectors changed less than 1 percentage point (table 1). Population growth was extending outward from the CBD with no major shifting of residents from one sector of the study area to another. The distribution of dwelling units followed the same general pattern as population.

Table 1.—Population, dwelling-unit, and density data for 1948 and 1955, grouped by rings and by sectors surrounding the central business district

Area	Population				Dwelling units				Number of dwelling units per residential acre, 1955 [1]
	1955		1948		1955		1948		
	Number	Percent	Number	Percent	Number	Percent	Number	Percent	
Rings:									
0–2 miles	268,256	18.4	271,468	24.5	105,494	21.8	89,357	26.5	65.7
2–4 miles	379,293	26.1	354,912	32.0	137,610	28.3	108,491	32.4	22.3
4–6 miles	433,848	29.8	305,163	27.5	134,789	27.8	86,126	25.6	10.2
6–8 miles	284,938	19.6	145,627	13.1	83,639	17.1	42,556	12.7	6.5
8–10 miles	88,102	6.1	32,681	2.9	24,176	5.0	9,657	2.8	4.7
Total area	1,454,437	100.0	1,109,851	100.0	485,108	100.0	336,181	100.0	12.5
Sectors:									
0	5,910	0.4	6,291	0.6	3,323	0.7	2,767	0.8	195.5
1	57,565	4.1	43,146	3.9	22,267	4.6	13,205	4.0	7.8
2	173,748	12.0	129,236	11.7	67,256	13.9	44,715	13.3	9.9
3	367,558	25.3	321,147	29.1	128,400	26.5	98,476	29.3	21.5
4	212,025	14.5	153,803	13.7	63,672	13.1	42,927	12.6	13.8
5	150,466	10.4	107,168	9.6	41,816	8.6	27,704	8.3	16.7
6	210,034	14.3	157,408	14.2	66,642	13.7	44,163	13.1	20.6
7	149,753	10.3	115,869	10.4	51,923	10.7	40,016	11.9	9.9
8	127,378	8.7	75,783	6.8	39,809	8.2	22,208	6.7	5.1
Total area	1,454,437	100.0	1,109,851	100.0	485,108	100.0	336,181	100.0	12.5

[1] No comparable data available for 1948.

ANALYSIS OF DISTANCE AND DIRECTION FACTORS

The volume of intra-area travel by residents in 1955 increased 42 percent over what it had been in 1948 (table 2). Trips to the CBD, on the other hand, increased only 4 percent, or from 255,338 to 265,659. In addition, residential (dwelling unit) growth had increased 44 percent (336,181 to 485,108), or at a rate slightly greater than trip growth (table 1). While this resulted in a less than 2-percent decrease in total area trips per dwelling unit, trips per 100 dwelling units destined to the CBD decreased 28 percent (76 to 55).

Of greatest significance was the decrease in the proportion of trips destined to the CBD. Although there was a 4-percent increase in the actual volume, the percentage relationship of these trips to total area travel had decreased from 14.9 to 10.9 percent which presents a picture of decreasing trip orientation to the CBD.

Since this study involves measurement of the effects of distance and direction from the central area on the travel patterns to the CBD, an analysis was made to determine the "degree of trip orientation" to the CBD of the trips made by residents of each ring-sector. This was accomplished by comparing the percentage of residents trips for each ring-sector destined to the CBD with the average percentage for the total area. Residents of ring-sectors whose percentage of trips to the CBD exceeded the average for the total area could be said to have a greater than total area average orientation to the CBD. On the other hand, residents of those ring-sectors having a percentage of trips to the CBD below the average would be considered less oriented than the area as a whole. The percentages were then ranked in class intervals of 0 to 20, 20 to 40, and 40 percent or more above or below the total area average for each of the 2 years. Figure 3 illustrates the relative orientation of trips to the CBD by residents of each ring-sector in the manner just described.

The degree of trip orientation was then com-

Table 2.—Trips to the central business district during 1948 and 1955, classified by purpose and mode of travel

Mode of travel	Trip comparisons	Work trips		Shopping trips		Other trips		All trips	
		1955	1948	1955	1948	1955	1948	1955	1948
Automobile driver	Total intra-area trips	272,097	164,109	108,458	45,847	777,700	418,824	1,158,255	628,780
	Trips to CBD	42,189	31,545	5,944	3,828	34,207	21,108	82,340	56,481
	Percentage of trips to CBD	15.5	19.2	5.5	8.3	4.4	5.0	7.1	9.0
	1955 percentage/1948 percentage	.81		.66		.88		.79	
	1955 trips to CBD/1948 trips to CBD	1.34		1.55		1.62		1.46	
Automobile passenger	Total intra-area trips	105,879	59,065	51,020	22,793	434,116	274,285	591,015	356,143
	Trips to CBD	33,419	19,125	4,472	3,581	12,552	12,010	50,443	34,716
	Percentage of trips to CBD	31.6	32.4	8.8	15.7	2.9	4.4	8.5	9.7
	1955 percentage/1948 percentage	.98		.56		.66		.88	
	1955 trips to CBD/1948 trips to CBD	1.75		1.25		1.05		1.45	
Taxi passenger	Total intra-area trips	13,550	10,017	2,974	1,901	41,572	34,558	58,096	46,476
	Trips to CBD	7,043	5,653	1,578	1,319	4,250	4,601	12,871	11,573
	Percentage of trips to CBD	52.0	56.4	53.1	69.4	10.2	13.3	22.2	24.9
	1955 percentage/1948 percentage	.92		.77		.77		.89	
	1955 trips to CBD/1948 trips to CBD	1.25		1.20		.92		1.11	
Mass-transit passenger	Total intra-area trips	183,857	219,003	29,765	39,619	408,060	419,576	621,682	678,198
	Trips to CBD	79,441	94,543	19,241	25,683	21,323	32,342	120,005	152,568
	Percentage of trips to CBD	43.2	43.2	64.6	64.8	5.2	7.7	19.3	22.5
	1955 percentage/1948 percentage	1.00		1.00		.68		.86	
	1955 trips to CBD/1948 trips to CBD	.84		.75		.66		.79	
All modes	Total intra-area trips	575,383	452,194	192,217	110,160	1,661,448	1,147,243	2,429,048	1,709,597
	Trips to CBD	162,092	150,866	31,235	34,411	72,332	70,061	265,659	255,338
	Percentage of trips to CBD	28.2	33.4	16.2	31.2	4.4	6.1	10.9	14.9
	1955 percentage/1948 percentage	.84		.52		.72		.73	
	1955 trips to CBD/1948 trips to CBD	1.07		.91		1.03		1.04	

pared for the 2 years to ascertain whether the general relationship of trips to the CBD had been maintained even though the percentage had decreased. In both years there were definite geographic areas which maintained a high degree of trip orientation. Residents who had a higher than average orientation in 1948, with few exceptions, retained this relative rank in 1955. Moreover, there were instances where the percentage of trips to the CBD decreased at a rate much less than that which was indicated by the average for the total area. As a result, these residents trips ranked even higher in relative orientation than they had in 1948. An example of this situation occurred in sectors 1, 2, and 3 (fig. 3).

Trip Purposes

The volume of residents intra-area work trips in 1955 increased 27 percent over that of 1948 (452,194 to 575,383). Work trips to the CBD, however, increased only 7 percent, from 150,866 to 162,092 (table 2). As a result, the percentage of work trips destined to the CBD decreased from 33.4 to 28.2 percent, or a 1955 to 1948 change ratio of 0.84. Although the CBD work trips decreased in relative importance since 1948, they had not decreased as much as shopping trips which had a change ratio of 0.52 or as much as "other" trips which had a change ratio of 0.72.

In each of the purpose categories, the proportion of CBD trips by mass transit decreased while trips by automobile increased (table 3). In actual volume, work trips by mass transit decreased from 94,543 to 79,441 (table 2). However, the 1955 to 1948 change ratio indicated that mass-transit work trips had an index of 1.00. The decreasing use of mass transit for travel to work was a general trend throughout the study area and not particularly characteristic of only the work trips to the CBD. This same characteristic applied to shopping trips to the CBD by mass transit.

The increasing proportion of work trips to the CBD by automobile passengers indicated a greater use of car pools. In the Washington study area, mounting congestion, increased costs in driving and parking, and less convenient transit service in suburban areas have done much to encourage the forming of car pools by residents who must travel to the CBD day after day.

The volume of shopping trips within the study area increased 74 percent between 1948 and 1955 (110,160 to 192,217 trips). In contrast, the number destined to the CBD decreased 9 percent from 34,411 to 31,235 trips (table 2). This decrease in the number of CBD shopping trips was reflected in the relative decline in the dollar volume of retail sales discussed earlier.

This decline in the volume of CBD shopping trips, however, does not so vividly illustrate the decreased importance of the CBD for shopping as does the comparison between 1948 and 1955 of the percentage of total area shopping trips destined to the CBD (31 percent in 1948 and 16 percent in 1955). This sizable decrease reflects the increasing development as well as the attraction of commercial concentrations outside the CBD.

The CBD shopping trip ratio of automobile passengers to that of automobile drivers decreased from an average of 94 passengers in 1948 to 75 passengers for every 100 automobile drivers in 1955. This was in contrast to the trend for work trips which increased from an average of 61 automobile passengers in 1948 to 79 for every 100 automobile drivers in 1955.

From these comparisons there is an indication of a decrease in the number of family trips made to the CBD for shopping. The physical layout and the facilities offered in the outlying shopping centers were more conducive to family shopping than those located in the CBD.

Since trips classified as "other" include a wide range of purposes, any comparisons of 1948 and 1955 data should be qualified. "Other" trips to the CBD increased 3 percent in 1955 over what they had been in 1948 (70,061 to 72,332). Compared to the 45-percent increase in the total intra-area trips for this group (table 2), the increase in trips to the CBD was small.

As was also found for work and shopping trips, the proportion of "other" trips to the CBD had decreased relative to the total number of "other" residents trips. This comparison applies to an average of a group of purposes, and some purposes were above and some below this average.

Notable for "other" trips to the CBD was the comparatively large increase in the volume of automobile-driver trips (62 percent) whereas there was a comparatively slight increase in the volume of automobile-passenger trips (5 percent). As in shopping trips, group and family trips to the CBD compared with those made to other parts of the study area appeared to be relatively less, and those that were made by an individual (such as a visit

Table 3.—Percentage distribution of trips to the central business district for various purposes in 1948 and 1955, classified by mode of travel

Purpose of trip	Automobile-driver trips		Automobile-passenger trips		Taxi-passenger trips		Mass-transit passenger trips		All trips	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Work	26.0	20.9	20.6	12.7	4.4	3.7	49.0	62.7	100.0	100.0
Ratio, 1955/1948	1.24		1.62		1.19		.78			
Shop	19.0	11.1	14.3	10.4	5.1	3.9	61.6	74.6	100.0	100.0
Ratio, 1955/1948	1.71		1.38		1.31		.83			
Other	47.3	30.1	17.3	17.1	5.9	6.6	29.5	46.2	100.0	100.0
Ratio, 1955/1948	1.57		1.01		.89		.64			
All purposes	31.0	22.1	19.0	13.6	4.8	4.5	45.2	59.8	100.0	100.0
Ratio, 1955/1948	1.40		1.40		1.07		.76			

) a doctor or a business trip) were apparently ıcreasing.

A separate geographical grouping of residents with a high degree of trip orientation was found for each of the three trip purposes studied (figs. 4–6). As indicated in figure 4, the largest continuous area of residents with an above average work trip orientation was located in both study years within sectors 1.

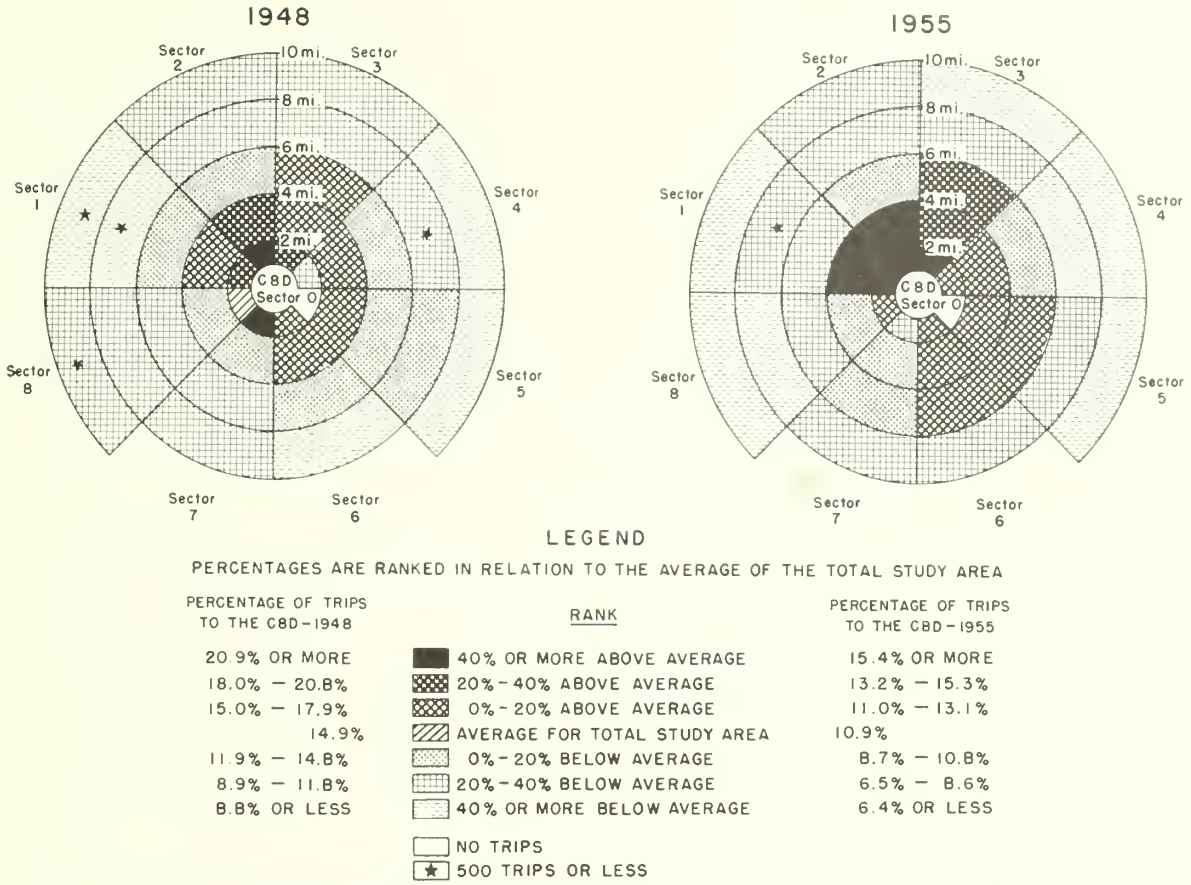


Figure 3.—Relation of CBD trips to total area trips for each ring-sector of residence.

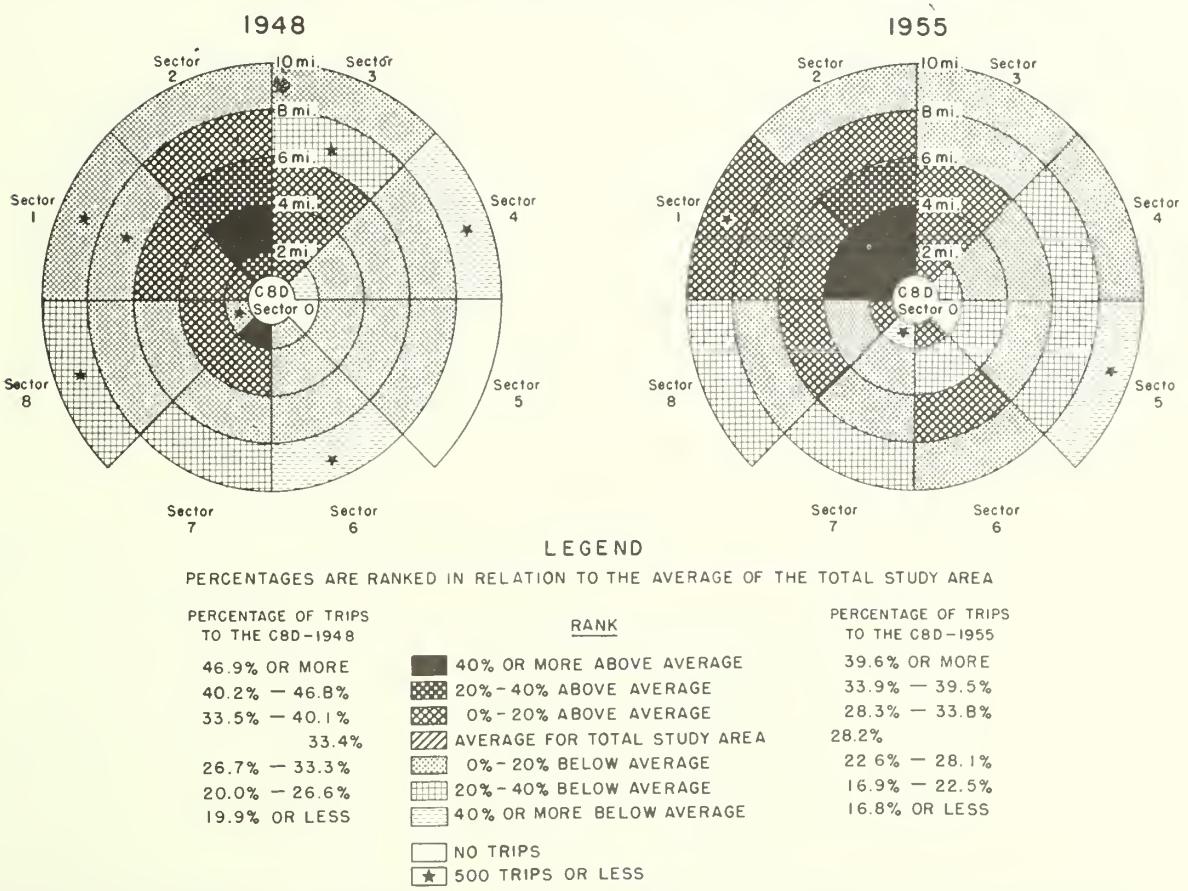
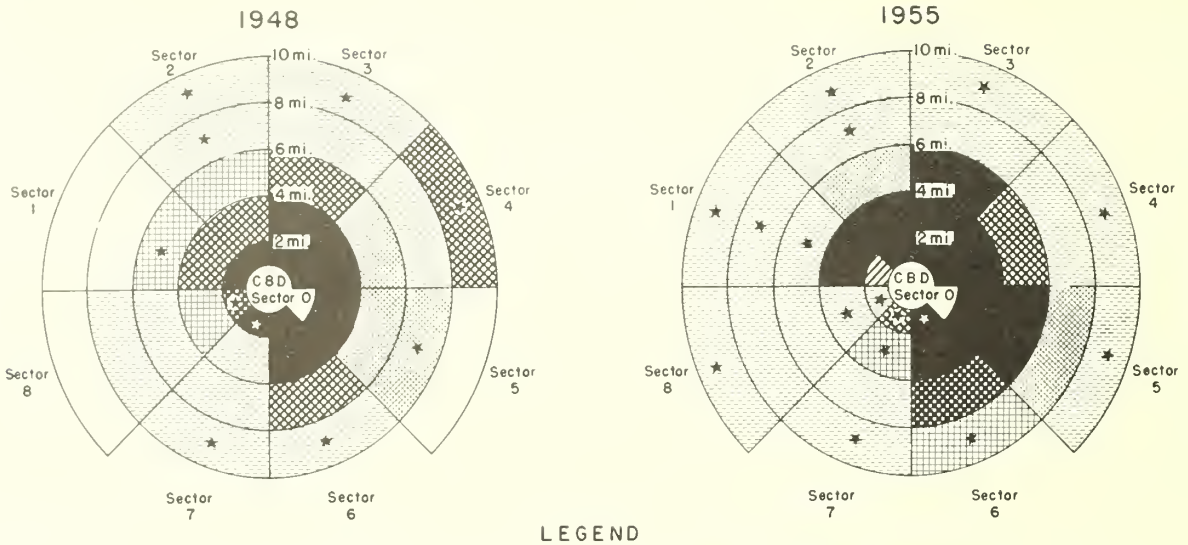


Figure 4.—Relation of CBD work trips to total area work trips for each ring-sector of residence.

2, and 3. On the other hand, figure 5 which shows the same information for shopping trips indicates that residents having an above average trip orientation for shopping were located within an entirely different geographical grouping. Hence, the degree of orientation varied between trip purposes within any one ring-sector.

Between 1948 and 1955 there was no major

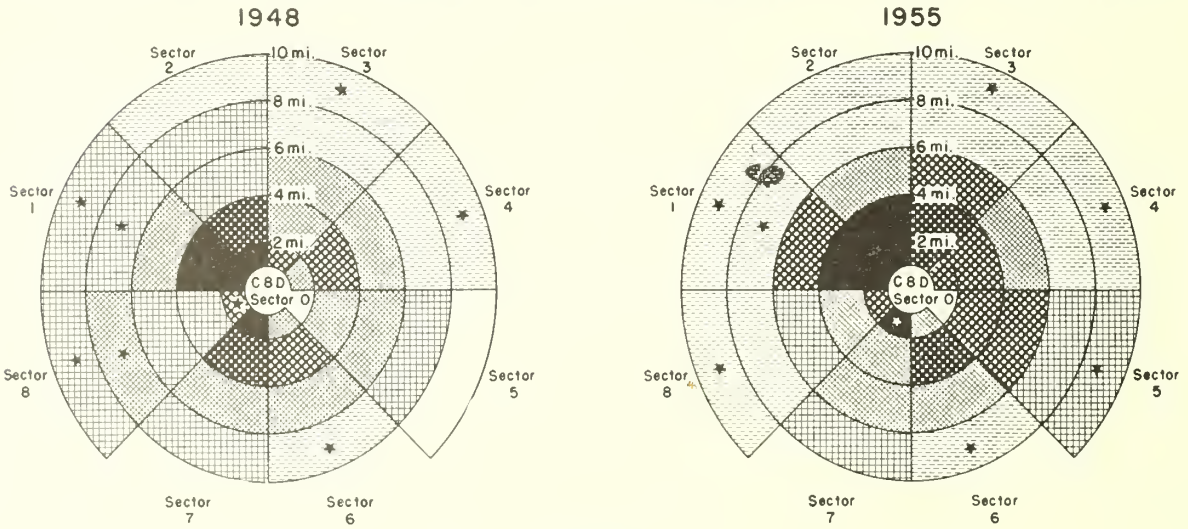


LEGEND

PERCENTAGES ARE RANKED IN RELATION TO THE AVERAGE OF THE TOTAL STUDY AREA

PERCENTAGE OF TRIPS TO THE CBD-1948	RANK	PERCENTAGE OF TRIPS TO THE CBD-1955
43.8% OR MORE	40% OR MORE ABOVE AVERAGE	22.8% OR MORE
37.5% — 43.7%	20% - 40% ABOVE AVERAGE	19.5% — 22.7%
31.3% — 37.4%	0% - 20% ABOVE AVERAGE	16.3% — 19.4%
31.2%	AVERAGE FOR TOTAL STUDY AREA	16.2%
25.0% — 31.1%	0% - 20% BELOW AVERAGE	13.0% — 16.1%
18.7% — 24.9%	20% - 40% BELOW AVERAGE	9.7% — 12.9
18.6% OR LESS	40% OR MORE BELOW AVERAGE	9.6% OR LESS
	NO TRIPS	
	★ 500 TRIPS OR LESS	

Figure 5.—Relation of CBD shopping trips to total area shopping trips for each ring-sector of residence.



LEGEND

PERCENTAGES ARE RANKED IN RELATION TO THE AVERAGE OF THE TOTAL STUDY AREA

PERCENTAGE OF TRIPS TO THE CBD-1948	RANK	PERCENTAGE OF TRIPS TO THE CBD-1955
8.6% OR MORE	40% OR MORE ABOVE AVERAGE	6.3% OR MORE
7.4% — 8.5%	20% - 40% ABOVE AVERAGE	5.4% — 6.2%
6.2% — 7.3%	0% - 20% ABOVE AVERAGE	4.5% — 5.3%
6.1%	AVERAGE FOR TOTAL STUDY AREA	4.4%
4.9% — 6.0%	0% - 20% BELOW AVERAGE	3.5% — 4.3%
3.7% — 4.8%	20% - 40% BELOW AVERAGE	2.6% — 3.4%
3.6% OR LESS	40% OR MORE BELOW AVERAGE	2.5% OR LESS
	NO TRIPS	
	★ 500 TRIPS OR LESS	

Figure 6.—Relation of "other" purpose CBD trips to total area "other" purpose trips for each ring-sector of residence.

Table 4.—Percentage distribution of trips to the central business district by various modes of travel in 1948 and 1955, classified by purpose

Mode of travel	Work trips		Shopping trips		Other trips		All trips	
	1955	1948	1955	1948	1955	1948	1955	1948
Automobile driver	51.2	55.9	7.2	6.8	41.6	37.3	100.0	100.0
Ratio, 1955/1948	.92	------	1.06	------	1.12	------	------	------
Automobile passenger	66.2	55.1	8.9	10.3	24.9	34.6	100.0	100.0
Ratio, 1955/1948	1.20	------	.86	------	.72	------	------	------
Taxi passenger	54.7	48.8	12.3	11.4	33.0	39.8	100.0	100.0
Ratio, 1955/1948	1.12	------	1.08	------	.83	------	------	------
Mass-transit passenger	66.2	62.0	16.0	16.8	17.8	21.2	100.0	100.0
Ratio, 1955/1948	1.07	------	.95	------	.84	------	------	------
All modes of travel	61.0	59.1	11.8	13.5	27.2	27.4	100.0	100.0
Ratio, 1955/1948	1.03	------	.87	------	.99	------	------	------

shifting of these geographical groupings of above-average orientation from one part of the study area to another, even though the percentage of trips destined to the CBD decreased for each purpose. There were, however, extensions of the relative rank of above-average orientation which had existed in 1948 among residents in ring-sectors closer to the CBD. In the case of shopping and "other" trips, no radial pattern of above-average orientation extended beyond a radius of 6 miles in 1955 as was the case for work trips.

Modes of Travel

The major change observed in residents choice of mode of travel to the CBD was the increased use of private automobiles and conversely the decreased use of mass transit (table 3). The volume of automobile-driver and automobile-passenger trips to the CBD in 1955 had approximately the same percentage increase over that of 1948, 46 and 45 percent, respectively (table 2). As has been pointed out, the increased proportion of automobile-passenger trips in the work category offset the lesser proportions in the other categories. The overall average of the number of automobile passengers to the number of automobile drivers traveling to the CBD remained approximately the same, 61.5 automobile passengers to every 100 automobile drivers in 1948 and 61.3 in 1955.

The volume of taxi-passenger trips to the CBD by residents increased 11 percent, from 11,573 to 12,871 trips (table 2).

There was a decrease in the percentage of trips destined to the CBD for each mode of travel. The mode of travel with the greatest relative change was automobile-driver trips, which had a change ratio of 0.79 (table 2). Even though the volume of automobile-driver trips to the CBD increased 46 percent over the period 1948 to 1955, trips to places outside the CBD had increased by an even greater percentage (88 percent).

The trip purposes for each travel mode indicate two significant characteristics (table 4). First, approximately one-half to two-thirds of the total volume of trips by each mode of travel in 1948 and 1955 were to work. Second, with the exception of automobile-driver trips, each of the modes of travel studied had a greater percentage of trips to work in 1955 and a smaller percentage for "other" purposes than was the case in 1948.

The increase in the percentage of automobile-driver trips made for "other" purposes reflects the increase in automobile ownership (61 cars per 100 dwelling units in 1948 and 79 in 1955). With the increased availability of an automobile as well as a general trend of an increase in the number of individuals licensed to drive, there was the tendency to drive to the luncheon engagement or to the night school in the CBD.

The degree of trip orientation that residents had in 1948 and 1955 indicated that there was a separate geographical grouping of ring-sector residents with above-average orientation for each of the modes used in traveling to the CBD (figs. 7–10). However, many of the radial patterns of above-average orientation, as seen in 1948 for each mode, were not so apparent in 1955. Residents beyond 6 miles showed a greater tendency to have a lower degree of trip orientation to the CBD in 1955 for each of the modes of travel than they had in 1948. Thus, the general relationship (rank) between ring-sector residents and the CBD, as related to the average percentage for each mode of travel, changed and a geographical regrouping took place rather than an extension of the relative rank of orientation, as in purpose of trip. The pattern in 1955 appeared to be more of a readjustment of travel, linked possibly with the growth of new centers of attraction outside the CBD as well as increased automobile ownership.

ANALYSIS OF DISTANCE FACTOR

The attitude of residents concerning distances from the central part of the city may change, even in so short a period as 7 years. A travel distance of 10 miles to the CBD in 1955 was not considered unusual, whereas in 1948 residents would have felt that the distance was too great. This changed attitude did not develop simultaneously in all directions from the CBD. Therefore, the averages used in discussing distance ring characteristics vary to a certain extent between sectors of a ring.

In this study of the effects of distance upon residents travel, several changes of a significant nature are apparent. The first, as seen in table 5, is a decrease in the actual volume of trips to the CBD by residents within 4 miles of the CBD and an increase in the number of residents trips beyond this distance. Secondly, as brought out in table 5, there was a decrease of approximately the same magnitude, 3 percent, in the proportion of trips destined to the CBD by residents of each of the rings.

The pattern in which the percentage of trips destined to the CBD decreased was also significant (table 5). In 1955 the percentage for each of the first three distance rings was approximately the same as the percentage for the next outer ring in 1948. For example, in 1955, residents within 2 miles of the CBD made 16.5 percent of their total trips to the CBD; this was approximately the same per-

Table 5.—Trips to the central business district in 1948 and 1955, classified according to purpose and distance of residence from the CBD

Trip purpose comparisons	0 to 2 miles		2 to 4 miles		4 to 6 miles		6 to 8 miles		8 to 10 miles		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Work trips:												
Total intra-area trips	101,239	107,962	161,175	155,674	175,317	125,071	106,331	51,452	31,321	12,035	575,383	452,194
Trips to CBD	31,944	37,387	47,592	54,504	50,363	41,508	24,256	14,153	7,937	3,314	162,092	150,866
Percentage of trips to CBD	31.6	34.6	29.5	35.0	28.7	33.2	22.8	27.5	25.3	27.5	28.2	33.4
1955 percentage/1948 percentage	.91	------	.84	------	.86	------	.83	------	.92	------	.84	------
1955 trips to CBD/1948 trips to CBD	.85	------	.87	------	1.21	------	1.71	------	2.39	------	1.07	------
Shopping trips:												
Total intra-area trips	16,557	13,599	36,452	31,674	66,744	42,297	52,801	18,483	19,663	4,107	192,217	110,160
Trips to CBD	7,545	8,936	10,062	12,040	9,725	9,907	3,337	2,847	566	681	31,235	34,411
Percentage of trips to CBD	45.6	65.7	27.6	38.0	14.6	23.4	6.3	15.4	2.9	16.6	16.2	31.2
1955 percentage/1948 percentage	.69	------	.73	------	.62	------	.41	------	.17	------	.52	------
1955 trips to CBD/1948 trips to CBD	.84	------	.84	------	.98	------	1.17	------	.83	------	.91	------
Other trips:												
Total intra-area trips	226,002	218,085	397,988	376,868	532,231	366,204	377,624	152,482	127,603	33,604	1,661,448	1,147,243
Trips to CBD	17,313	19,751	21,914	25,394	21,461	18,467	8,912	5,586	2,732	863	72,332	70,061
Percentage of trips to CBD	7.7	9.1	5.5	6.7	4.0	5.0	2.4	3.7	2.1	2.6	4.4	6.1
1955 percentage/1948 percentage	.85	------	.82	------	.80	------	.65	------	.81	------	.72	------
1955 trips to CBD/1948 trips to CBD	.88	------	.86	------	1.16	------	1.60	------	3.17	------	1.03	------
All purposes:												
Total intra-area trips	343,798	339,646	595,615	564,216	774,292	533,572	536,756	222,417	178,587	49,746	2,429,048	1,709,597
Trips to CBD	56,802	66,074	79,568	91,938	81,549	69,882	36,505	22,586	11,235	4,858	265,659	255,338
Percentage of trips to CBD	16.5	19.5	13.4	16.3	10.5	13.1	6.8	10.2	6.3	9.8	10.9	14.9
1955 percentage/1948 percentage	.85	------	.82	------	.80	------	.67	------	.64	------	.73	------
1955 trips to CBD/1948 trips to CBD	.86	------	.87	------	1.17	------	1.62	------	2.31	------	1.04	------

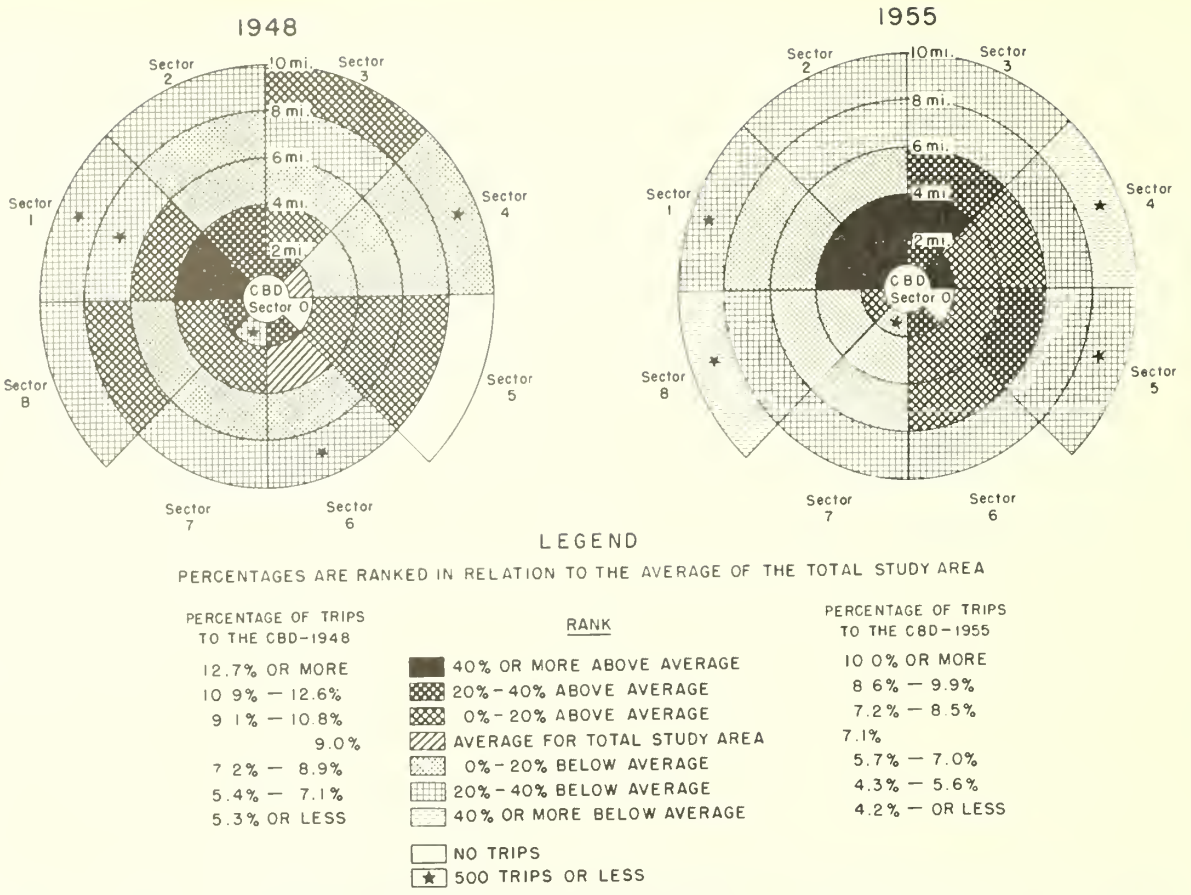


Figure 7.—Relation of CBD automobile-driver trips to total area automobile-driver trips for each ring-sector of residence.

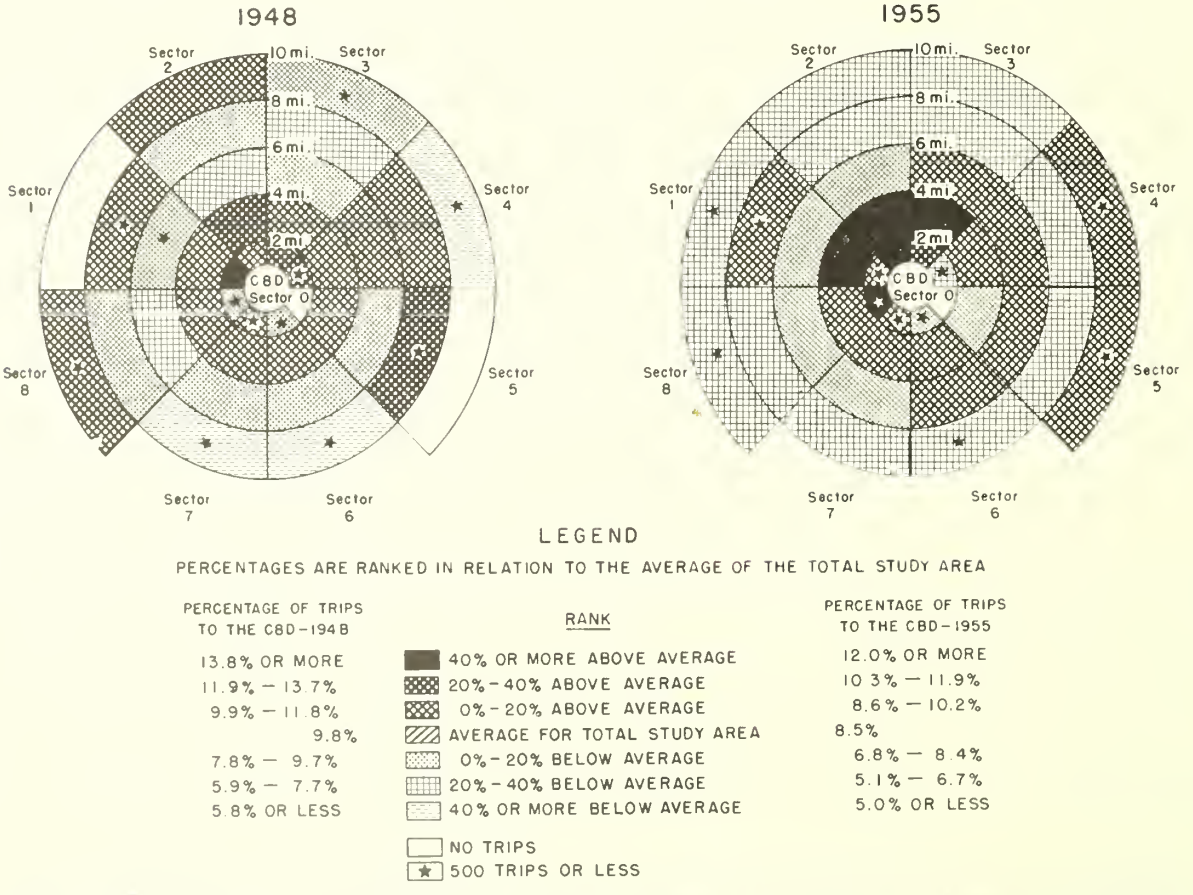
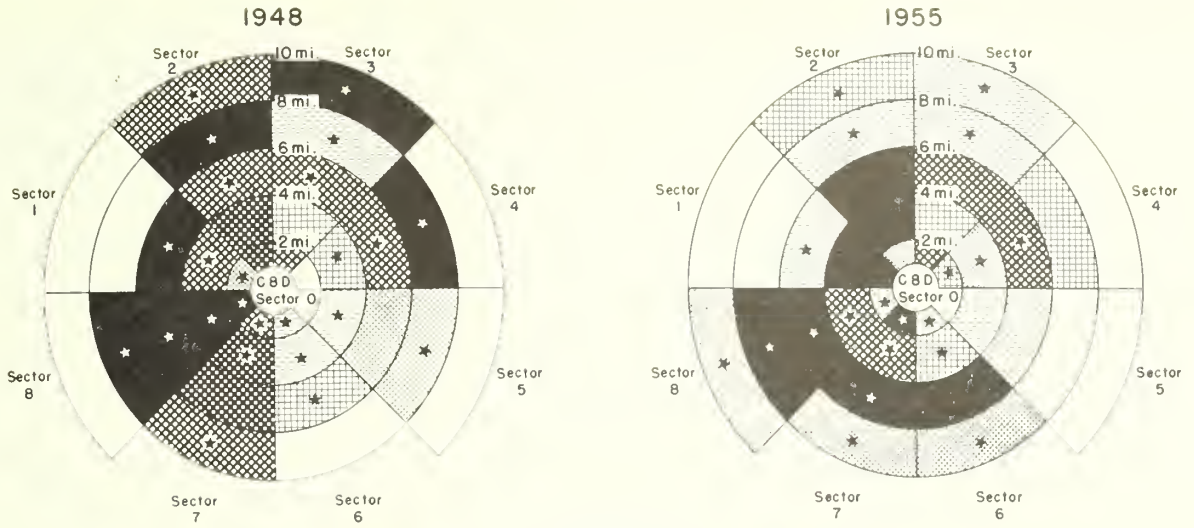


Figure 8.—Relation of CBD automobile-passenger trips to total area automobile-passenger trips for each ring-sector of residence.



LEGEND

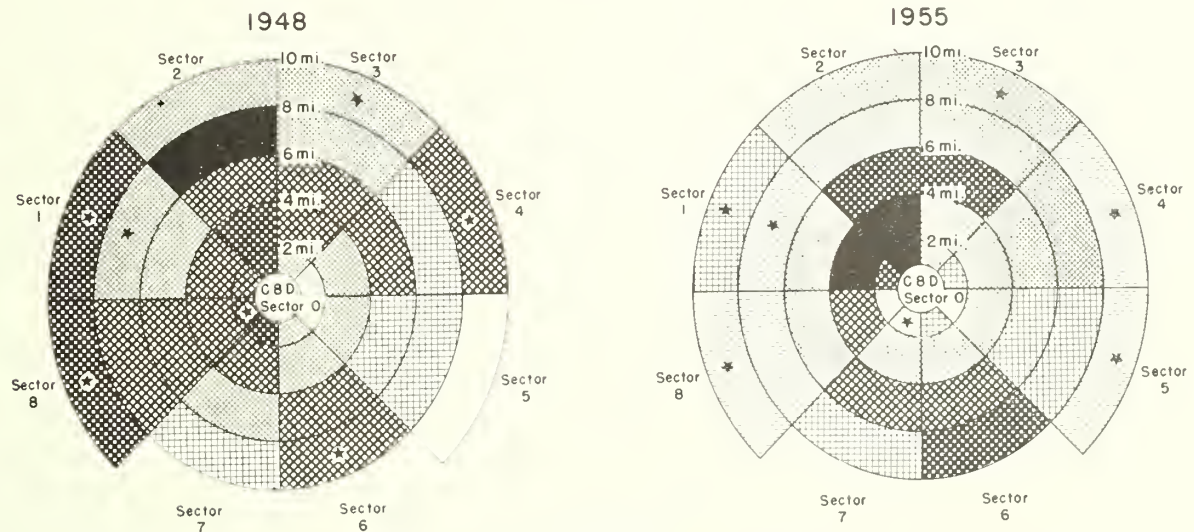
PERCENTAGES ARE RANKED IN RELATION TO THE AVERAGE OF THE TOTAL STUDY AREA

PERCENTAGE OF TRIPS TO THE CBD–1948	RANK	PERCENTAGE OF TRIPS TO THE CBD–1955
35.0% OR MORE	40% OR MORE ABOVE AVERAGE	31.2% OR MORE
30.0% — 34.9%	20% – 40% ABOVE AVERAGE	26.7% — 31.1%
25.0% — 29.9%	0% – 20% ABOVE AVERAGE	22.3% — 26.6%
24.9%	AVERAGE FOR TOTAL STUDY AREA	22.2%
19.9% — 24.8%	0% – 20% BELOW AVERAGE	17.8% — 22.1%
14.9% — 19.8%	20% – 40% BELOW AVERAGE	13.3% — 17.7%
14.8% OR LESS	40% OR MORE BELOW AVERAGE	13.2% OR LESS

NO TRIPS

500 TRIPS OR LESS

Figure 9.—Relation of CBD taxi-passenger trips to total area taxi-passenger trips for each ring-sector of residence.



LEGEND

PERCENTAGES ARE RANKED IN RELATION TO THE AVERAGE OF THE TOTAL STUDY AREA

PERCENTAGE OF TRIPS TO THE CBD–1948	RANK	PERCENTAGE OF TRIPS TO THE CBD–1955
31.6% OR MORE	40% OR MORE ABOVE AVERAGE	27.1% OR MORE
27.1% — 31.5%	20% – 40% ABOVE AVERAGE	23.3% — 27.0%
22.6% — 27.0%	0% – 20% ABOVE AVERAGE	19.4% — 23.2%
22.5%	AVERAGE FOR TOTAL STUDY AREA	19.3%
18.0% — 22.4%	0% – 20% BELOW AVERAGE	15.4% — 19.2%
13.5% — 17.9%	20% – 40% BELOW AVERAGE	11.6% — 15.3%
13.4% OR LESS	40% OR MORE BELOW AVERAGE	11.5% OR LESS

NO TRIPS

500 TRIPS OR LESS

Figure 10.—Relation of CBD mass-transit passenger trips to total area mass-transit passenger trips for each ring-sector of residence.

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Table 6.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to purpose and distance of residence from the CBD

Distances from CBD	Work trips		Shopping trips		Other trips		All purposes	
	1955	1948	1955	1948	1955	1948	1955	1948
0–2 miles	19.7	24.8	24.2	26.0	23.9	28.2	21.4	25.9
2–4 miles	29.3	36.1	32.2	35.0	30.3	36.2	30.0	36.0
4–6 miles	31.1	27.5	31.1	28.7	29.7	26.4	30.7	27.4
6–8 miles	15.0	9.4	10.7	8.3	12.3	8.0	13.7	8.8
8–10 miles	4.9	2.2	1.8	2.0	3.8	1.2	4.2	1.9
Total area	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

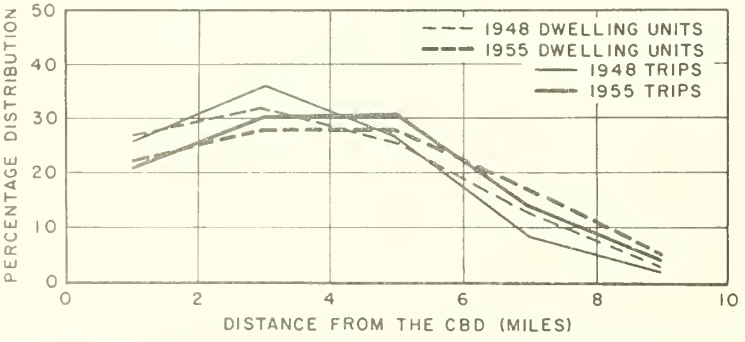

Figure 11.—Percentage distribution of residents CBD trips and dwelling units, according to distance from the CBD.

centage as for residents within 2 to 4 miles of the CBD in 1948. If this trend were to continue for the following 7 years, the residents within 2 miles of the CBD in 1962 would make approximately the same percentage of trips to the CBD as residents 2 to 4 miles from the CBD made in 1955, or 13 percent; residents 2 to 4 miles and 4 to 6 miles from the CBD would make 10 and 6 percent, respectively, of their trips to the CBD in 1962.

The percentage of trips to the CBD by residents 6 to 8 and 8 to 10 miles from the CBD did not fit the pattern of change just described. The percentage for both rings was approximately the same in each year. This similarity in the two outer rings, which are basically suburban, suggests that residents of suburban areas tend to have a more similar orientation to the CBD than urban areas closer to the CBD.

A significant factor in the distribution by distance of residents CBD trips was the leveling off which occurred between 2 to 4 miles and 4 to 6 miles of the CBD (table 6 and figure 11), a characteristic also observed in the distribution of dwelling units. In 1948, the major proportion of trips to the CBD were made by residents within 2 to 4 miles of the CBD. At that time, residents of this ring accounted for 91,938 or 36 percent of the total trips to the CBD. In 1955, however, the number of CBD trips made by residents within 2 to 4 miles had decreased and similar trips made by residents within 4 to 6 miles had increased. As a result, the number of trips from the two rings were about the same, 79,568 and 81,549, respectively.

This characteristic of residents trips to the CBD is important in estimating future trip distribution. Theoretically, it is altogether possible that total trips to the CBD made by residents of each of the five distance rings could approach, although not reach, a more uniform distribution. Over a period of time the level of trip distribution found between the 2- to 4- and 4- to 6-mile rings would just extend outward at a lower percentage level through the 6- to 8- and possibly the 8- to 10-mile rings. As the percentage of CBD trips by residents residing between 6 and 10 miles of the CBD increases, there will be a relative decrease in the percentage of the total CBD trips made by residents within 6 miles of the CBD.

Two observations support this possible trend in residents trip distribution. The first was the greater uniformity in the number of CBD trips per 100 dwelling units throughout the five distance rings in 1955 (table 7); the range in the number of trips was from 44 to 61, or a difference of 17. In 1948, the range in the number of trips among distance rings was from 50 to 85, or a difference of 35.

The second observation has to do with the increase in the proportion of dwelling units located between 6 and 10 miles of the CBD (table 1). In 1948, this area contained 15.5 percent of the total dwelling units within the study area; by 1955, the proportion had increased to 22.1 percent. Each consecutive ring circumscribed a much larger area than the preceding ring. Even though residential density per acre is progressively less as distance from the CBD increases, the larger area of each consecutive ring permits a corresponding increase in the number of residences within each ring when fully developed. This has occurred in the 2- to 4- and 4- to 6-mile distance rings. In 1955, the densities within these rings were 22.3 and 10.2 dwelling units per acre, respectively, but the total number of residences for the 2 rings were nearly equal, 137,610 and 134,789

If there continues to be an increasing similarity in the frequency of CBD trips per 100 dwelling units, and if it is assumed that the residential development within 6 miles of the CBD will become more static as less land is available for new development, and the residential development between 6 and 10 miles of the CBD continues at a high rate of increase because of the open land available, then, theoretically, there would be a greater increase in the number of trips from the outer rings than from those closer to the city center. The percentage of total CBD trips by residents located between 6 and 10 miles of the CBD will increase, and this would cause a relative decrease in the proportion of total CBD trips made by residents within 6 miles of the CBD.

Trip Purposes

Between 1948 and 1955, a change took place in the percentage distribution of residents trip purposes that was related to distance from the CBD (table 8). In 1948 the proportion of residents trips to the CBD for work increased with distance from the CBD. The reverse was true of "other" trips. The proportion of trips for shopping, however, varied only slightly with distance; the difference between the nearest ring and the farthest ring from the CBD was only 0.5 percent.

The pattern of trip distribution in 1955 remained about the same as 1948 for work and "other" trips. In contrast, the percentage of CBD trips for shopping decreased with distance, from 13.3 percent within 2 miles of the CBD to 5.0 percent within 8 to 10 miles of the CBD. This comparison shows the effect of increased commercial development outside the CBD, especially in the suburban communities.

Work trips

In 1955 there was no concentration of residents making work trips to the CBD in any single concentric distance ring as was the case in 1948. As shown in table 6 for 1948, residents 2 to 4 miles from the CBD had made the largest proportion of the total work trips destined to the CBD (36.1 percent). By 1955, however, the percentage distribution had

Table 7.—Number of trips per 100 dwelling units to the central business district in 1948 and 1955, classified according to purpose and distance of residence from the CBD

Distances from CBD	Work trips		Shopping trips		Other trips		All purposes	
	1955	1948	1955	1948	1955	1948	1955	1948
0–2 miles	30.3	41.8	7.2	10.0	16.4	22.1	53.8	73.9
2–4 miles	34.6	50.2	7.3	11.1	15.9	23.4	57.8	84.7
4–6 miles	37.4	48.2	7.2	11.5	15.9	23.4	60.5	81.1
6–8 miles	29.2	33.3	4.0	6.7	10.7	13.1	44.0	53.1
8–10 miles	32.8	34.3	2.3	7.1	11.3	8.9	46.5	50.3
Average, total area	33.4	44.9	6.4	10.2	14.9	20.8	54.8	76.0

changed, and there was only a 2-percent difference between the proportion of trips made by the residents within 2 to 4 and 4 to 6 miles of the CBD (29.3 and 31.1 percent, respectively). The changes that occurred during the 7-year period are illustrated graphically in figure 12. As work trips made up a major portion of the total CBD trips, it is apparent that residents work trips were an important factor in the leveling of the distribution curve.

In the future an increasing percentage of CBD work trips is expected to be made by residents of the outer rings, extending the plateau of percentage distribution at a lower percentage level over an even wider area than in 1955. As shown in table 7, the range among distance rings for work trips destined to the CBD per 100 dwelling units was only half as great in 1955 as it was in 1948. If this range of difference were to be maintained

Table 8.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to trip purpose of mode traveled and distance of residence from the CBD

Mode of travel and trip purpose	0 to 2 miles		2 to 4 miles		4 to 6 miles		6 to 8 miles		8 to 10 miles		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Automobile-driver trips to—												
Work	45.0	48.4	46.4	54.8	50.3	55.2	58.4	64.0	66.6	70.1	51.2	55.8
Ratio, 1955/1948	.93		.85		.91		.91		.95		.92	
Shop	3.8	6.5	8.7	5.9	8.2	7.4	7.4	6.9	2.1	9.2	7.2	6.8
Ratio, 1955/1948	.58		1.47		1.11		1.07		.23		1.06	
Other	51.2	45.1	44.9	39.3	41.5	37.4	34.2	29.1	31.3	20.7	41.6	37.4
Ratio, 1955/1948	1.14		1.14		1.11		1.18		1.51		1.11	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Automobile-passenger trips to—												
Work	54.5	51.6	63.0	52.7	68.0	56.8	72.8	61.2	75.0	56.2	66.2	55.1
Ratio, 1955/1948	1.06		1.20		1.20		1.19		1.33		1.20	
Shop	12.0	6.1	8.7	10.3	9.4	11.7	7.3	9.6	5.3	20.1	8.9	10.3
Ratio, 1955/1948	1.97		.84		.80		.76		.26		.86	
Other	33.5	42.3	28.3	37.0	22.6	31.5	19.9	29.2	19.7	23.7	24.9	34.6
Ratio, 1955/1948	.79		.76		.72		.68		.83		.72	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Taxi-passenger trips to—												
Work	56.1	51.3	56.8	47.1	50.1	48.7	57.7	30.7	33.5	87.7	54.7	48.8
Ratio, 1955/1948	1.09		1.21		1.03		1.88		.38		1.12	
Shop	9.1	10.4	11.6	10.4	18.8	14.1	11.6	17.3	8.4		12.3	11.4
Ratio, 1955/1948	.88		1.12		1.33		.67				1.08	
Other	34.8	38.3	31.6	42.5	31.1	37.2	30.7	52.0	58.1	12.3	33.0	39.8
Ratio, 1955/1948	.91		.74		.84		.59		4.72		.83	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Mass-transit passenger trips to—												
Work	60.0	59.0	65.9	63.0	70.3	63.0	73.2	64.3	76.0	74.2	66.2	62.0
Ratio, 1955/1948	1.02		1.05		1.12		1.14		1.02		1.07	
Shop	17.0	15.9	16.1	16.3	16.1	18.5	13.4	18.4	11.1	17.7	16.0	16.8
Ratio, 1955/1948	1.07		.99		.87		.73		.63		.95	
Other	23.0	25.1	18.0	20.7	13.6	18.5	13.4	17.3	12.9	8.1	17.8	21.2
Ratio, 1955/1948	.92		.87		.74		.77		1.59		.84	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
All modes of travel to—												
Work	56.2	56.6	59.8	59.3	61.8	59.4	66.4	62.7	70.7	68.2	61.0	59.1
Ratio, 1955/1948	.99		1.01		1.04		1.06		1.04		1.03	
Shop	13.3	13.5	12.7	13.1	11.9	14.2	9.2	12.6	5.0	14.0	11.8	13.5
Ratio, 1955/1948	.99		.97		.84		.73		.36		.87	
Other	30.5	29.9	27.5	27.6	26.3	26.4	24.4	24.7	24.3	17.8	27.2	27.4
Ratio, 1955/1948	1.02		1.00		1.00		.99		1.37		.99	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 9.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to mode of travel of trip purpose and distance of residence from the CBD

Trip purpose and mode of travel	0 to 2 miles		2 to 4 miles		4 to 6 miles		6 to 8 miles		8 to 10 miles		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Work trips by—												
Automobile drivers	14.7	10.3	20.5	19.4	29.8	25.7	38.0	35.1	44.3	44.4	26.0	20.9
Ratio, 1955/1948	1.43		1.06		1.16		1.08		1.00		1.24	
Automobile passengers	11.7	7.7	17.7	11.7	22.8	15.2	29.3	20.1	33.4	21.4	20.6	12.7
Ratio, 1955/1948	1.52		1.51		1.50		1.46		1.56		1.62	
Taxi passengers	8.8	6.5	4.9	3.2	3.0	2.8	1.4	1.3	.7	4.5	4.4	3.7
Ratio, 1955/1948	1.35		1.53		1.07		1.08		.16		1.19	
Mass-transit passengers	64.8	75.5	56.9	65.7	44.4	56.3	31.3	43.5	21.6	29.7	49.0	62.7
Ratio, 1955/1948	.86		.87		.79		.72		.73		.78	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Shopping trips by—												
Automobile drivers	5.3	5.8	18.1	9.5	25.1	14.5	35.2	18.9	20.0	28.2	19.0	11.1
Ratio, 1955/1948	.91		1.91		1.73		1.86		.71		1.71	
Automobile passengers	10.9	3.8	11.6	10.4	16.3	13.0	21.3	15.7	33.4	37.3	14.3	10.4
Ratio, 1955/1948	2.87		1.12		1.25		1.36		.90		1.38	
Taxi passengers	6.0	5.5	4.8	3.2	5.8	3.4	2.0	3.7	2.3		5.1	3.9
Ratio, 1955/1948	1.09		1.50		1.71		.54				1.31	
Mass-transit passengers	77.8	84.9	65.5	76.9	52.8	69.1	41.5	61.7	44.3	34.5	61.6	74.6
Ratio, 1955/1948	.92		.85		.76		.67		1.28		.83	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Other trips by—												
Automobile drivers	30.8	18.2	43.0	29.9	57.8	39.2	60.6	40.4	60.5	50.4	47.3	30.1
Ratio, 1955/1948	1.69		1.44		1.47		1.50		1.20		1.57	
Automobile passengers	13.3	12.0	17.3	17.7	17.8	18.8	21.8	24.3	25.5	34.8	17.3	17.1
Ratio, 1955/1948	1.11		.98		.95		.90		.73		1.01	
Taxi passengers	10.1	9.1	6.0	6.2	4.3	4.8	2.0	5.8	3.3	2.4	5.9	6.6
Ratio, 1955/1948	1.10		.97		.90		.34		1.38		.89	
Mass-transit passengers	45.8	60.7	33.7	46.2	20.1	37.2	15.6	29.5	10.7	12.4	29.5	46.2
Ratio, 1955/1948	.75		.73		.54		.53		.86		.64	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
All purposes by—												
Automobile drivers	18.4	12.0	26.4	21.0	36.6	27.7	43.2	34.4	47.0	43.2	31.0	22.1
Ratio, 1955/1948	1.53		1.26		1.32		1.26		1.09		1.40	
Automobile passengers	12.1	8.5	16.9	13.2	20.7	15.8	26.7	20.6	31.5	26.0	19.0	13.6
Ratio, 1955/1948	1.42		1.28		1.31		1.30		1.21		1.40	
Taxi passengers	8.8	7.1	5.1	4.0	3.7	3.4	1.6	2.7	1.4	3.5	4.8	4.5
Ratio, 1955/1948	1.24		1.28		1.09		.59		.40		1.07	
Mass-transit passengers	60.7	72.4	51.6	61.8	39.0	53.1	28.5	42.3	20.1	27.3	45.2	59.8
Ratio, 1955/1948	.84		.83		.73		.67		.74		.76	
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

or decreased even more, then those rings with the greatest future growth in residential development would tend to have the largest increases in residents work trips to the CBD.

On the basis of present day residential development, it is expected that the majority of future dwelling units will be constructed in the outer rings, thereby increasing the relative percentage of total work trips to the CBD from these areas. This is already apparent in the increased proportion of the total work trips made to the CBD by residents of the 6- to 8-mile ring. In 1948, residents of this ring made 9.4 percent of the CBD work trips as compared with 15.0 percent in 1955.

Although the volume of residents work trips destined to the CBD in 1955 decreased within 4 miles of the CBD and increased beyond this distance, the percentage made to the CBD by residents of each distance ring decreased (table 5). It should be noted also that trips made by residents within 2 miles and within 8 to 10 miles of the CBD had approximately the same change ratio, 0.91 and 0.92, respectively. Because of distance, it would be expected that a greater decrease in residents trips would be found in the 8- to 10-mile ring than in the area between 2 and 8 miles.

The explanation of this relationship was found by examining the trips made by residents of sector 2 of the 8- to 10-mile ring. Residents of this sector, as will be mentioned later, had an above-average orientation of intra-area work trips that were destined to the CBD. Residents of this ring-sector also accounted for over one-half of the trips made by residents of this ring. Because of the small number of trips made by residents of the other sectors of this ring, residents of sector 2 greatly influenced the results for this ring. It is possible that as residential development increases in the other sectors of the 8- to 10-mile ring, that are less oriented to the CBD than sector 2, a change ratio more in line with the travel distance will occur.

Another change which took place during the 7-year period was the shift in mode of travel to work (table 9). For both years, the general pattern was for the percentages of automobile-driver and automobile-passenger work trips to increase with distance from the CBD while the percentages of work trips made by taxi passengers and mass-transit passengers decreased with distance. However, in 1955, the most significant difference was the decreasing distance in which mass transit was the dominant mode used to travel to work. In 1948 residents within 6 miles of the central area made 50 percent or more of their work trips to the CBD by mass transit. In 1955 this percentage applied to a 4-mile radius.

It should be noted that the percentage of residents work trips made by mass transit in 1955 for any given distance ring was approximately the same as the percentage in 1948 for the next outer ring. For example, in 1955, residents within 2 miles of the CBD made 64.8 percent of their work trips by mass transit; in 1948 residents within 2 to 4 miles made approximately the same percentage, 65.7. This same relationship follows regardless of distance ring. If this same pattern were to continue in the 7 years following 1955, then by 1962 only the residents within 2 miles of the CBD would make over 50 percent of their work trips to the CBD by mass transit.

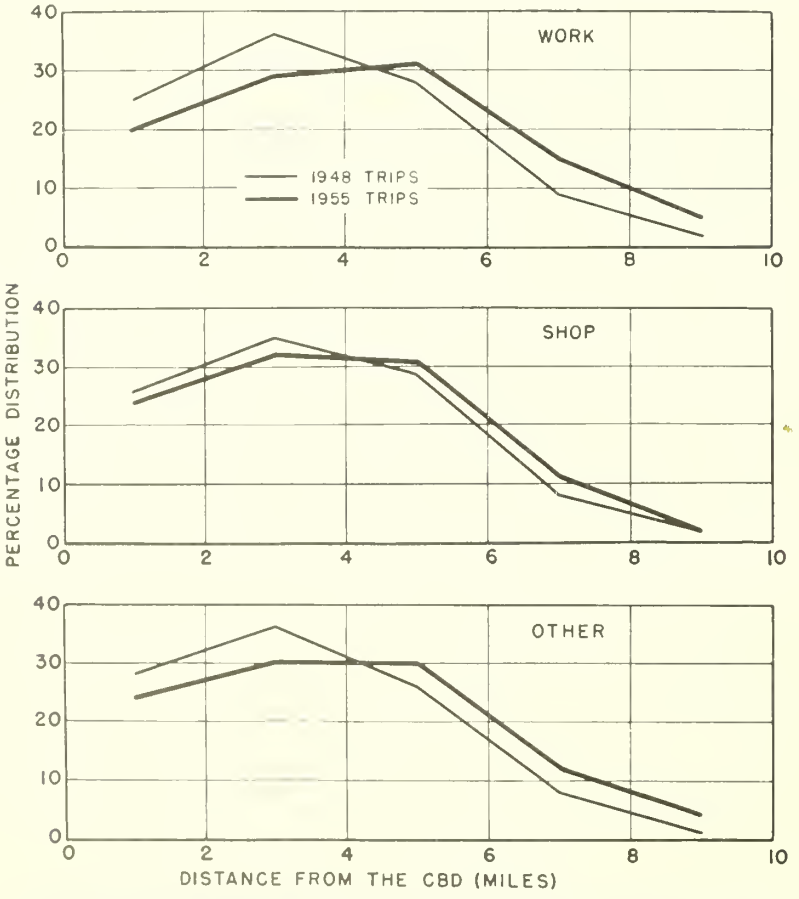

Figure 12.—Percentage distribution of work, shop, and other purpose trips to the CBD, according to distance from the CBD.

Shopping trips

The percentage distribution of residents shopping trips to the CBD changed less between the five distance rings than the distribution of work or "other" trips, even though there was a decrease in the total volume of shopping trips to the CBD between 1948 and 1955. Of the relative changes that did take place, however, the most significant was the apparent leveling off of the peak of trip distribution within 2 to 4 miles of the CBD (fig. 12). Residents within 2 to 4 and 4 to 6 miles of the CBD in 1955 made 32.2 and 31.1 percent, respectively, of the shopping trips to the CBD (table 6).

In the previous discussion of work trips it was mentioned that with growth in residential development beyond 6 miles of the CBD, the leveling of the distribution curve would likely extend over an even w der area than in 1955, and this would result in a more uniform distribution of residents work trips to the CBD. In the case of shopping trips, however, there is no basis for such speculation.

Between 1948 and 1955 the average number of daily shopping trips to the CBD per 100 dwelling units decreased in each of the 2-mile rings, and the percentage decrease became progressively greater with distance from the central area (table 7). It is evident that residents, particularly those within the first three distance rings, were patronizing stores outside the CBD for many of their needs and relied on the CBD stores for shopping goods merchandise or for items where a greater selection was desired. In the case of the outlying communities (beyond 6 miles of the CBD), residents had ready access to major shopping areas which could supply practically all of their needs. Two department stores had a suburban branch in 1948, and these were relatively small in sales floor area. By 1955, one or more branches had been established by each of the large department stores and by many of the large specialty stores.

In 1948 the area between 6 and 10 miles of the CBD contained approximately 52,200 dwelling units and accounted for a total of 22,600 intra-area shopping trips; of this number, 3,500 were made to the CBD. By 1955 the number of dwelling units had increased to 107,200 and intra-area shopping trips had increased to 72,500; yet, of these trips, only 3,900 were to the CBD. The percentage of intra-area shopping trips destined to the CBD decreased during the 7-year period from 15.6 to 5.4 percent, a change ratio of 0.35.

Of the changes in mode of travel, the most significant was the decreased distance from the CBD in which mass transit accounted for one-half or more of residents shopping trips to the CBD (table 9). In 1948 residents within 8 miles of the CBD used mass transit for more than half of their trips; in 1955, the

same ratio applied only to residents within 6 miles.

Mass-transit passenger travel, although decreasing in the relative percentage of total shopping trips destined to the CBD, nevertheless accounted for about three-fourths of all shopping trips made to the CBD by residents within a 2-mile radius and two-thirds of such trips by residents within 2 to 4 miles of the CBD. These high percentages were maintained because of the large number of well distributed, converging transit lines which provided the more densely populated rings with convenient passenger service to the retail core of the CBD.

The only group to show an increase between 1948 and 1955 in the percentage of shopping trips by mass transit were residents of the 8- to 10-mile ring. This apparent inconsistency may be due to sample variability, as the total number of shopping trips to the downtown area by residents of this ring was relatively small, the calculated figure being 681 in 1948 and 566 in 1955. Since the sampling rate was 1 in 20 in this area in 1948 and 1 in 10 in 1955, the number of residents interviewed was only 34 in the first year and 56 in the second year. When broken down by mode of travel, therefore, the reliability of the data on mode of travel for this ring is not great.

Table 10.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to mode of travel and distance of residence from the CBD

Distances from CBD	Automobile driver		Automobile passenger		Taxi passenger		Mass-transit passenger	
	1955	1948	1955	1948	1955	1948	1955	1948
0–2 miles	12.6	14.1	13.6	16.2	39.0	40.6	28.8	31.3
2–4 miles	25.5	34.2	26.6	35.0	32.2	31.9	34.2	37.2
4–6 miles	36.3	34.3	33.5	31.8	23.2	20.6	26.5	24.3
6–8 miles	19.2	13.7	19.3	13.4	4.4	5.4	8.6	6.3
8–10 miles	6.4	3.7	7.0	3.6	1.2	1.5	1.9	.9
Total area	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

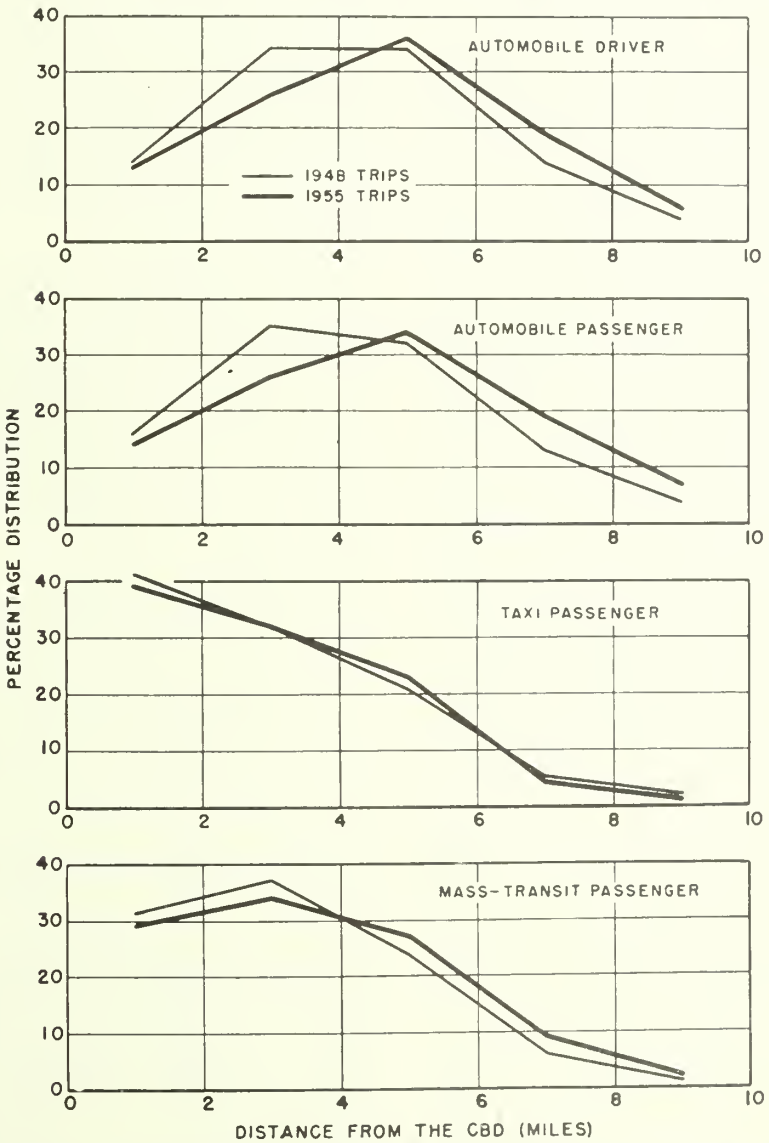

Figure 13.—Percentage distribution of automobile-driver, automobile-passenger, taxi-passenger, and mass-transit passenger trips to the CBD, according to distance from the CBD.

Other purpose trips

The volume of CBD trips made for purposes other than to work or shop by residents within a 4-mile radius of the CBD decreased between 1948 and 1955. Beyond 4 miles, however, there was an increase in the number of such trips. The distribution pattern followed closely that of work and shopping trips (table 6 and figure 12).

Because of the diverse purposes included in the "other" category, it is difficult to do any more than speculate on the future position of the distribution curve. As a group, however, these trip purposes had several of the trend characteristics observed in the distribution of work trips. First, the range between the ring with the largest and the one with the smallest number of "other" trips to the CBD per 100 dwelling units decreased from 14 trips in 1948 to 6 in 1955 (table 7), and secondly, residents beyond 6 miles were making an increasing proportion of total "other" trips to the CBD (table 6).

With increased residential development in the outer rings and decreasing differences between rings in trips per dwelling unit, the distribution curve over a period of time might be expected to parallel that of work trips. In any discussion of future trends, the several purpose categories that make up "other" trips must be considered. Trips for purposes of social-recreation, eat meal, and business are not quite so rigid in their directional movement, and there is greater opportunity for choice of destination than there is for work trips.

The percentage of total intra-area "other" trips that were made to the CBD decreased for residents of each ring (table 5). With the exception of trips made by residents 8 to 10 miles of the CBD, the percentage change indicated by the change ratio shown in table 5 increased with distance. In the case of the residents within the 8- to 10-mile ring, the proportion of intra-area trips made to the CBD was already very small.

Within each ring the change in the distribution pattern of travel modes to the CBD for "other" purposes varied somewhat from that which was indicated for work and shopping purposes. In neither of the latter two purposes did any of the separate modes of travel other than mass-transit passenger have a percentage of residents trips to the CBD greater than 50 percent in any of the distance rings. Automobile-driver trips to the CBD, however, were the dominant mode of travel by residents 8 to 10 miles of the CBD in 1948, and this same characteristic applied to residents beyond 4 miles of the CBD in 1955.

It was previously noted that for working and shopping trips there was a reduction during the 7-year period in the area in which half or more of the CBD trips were made by mass transit. In the case of "other" purposes, only residents within 2 miles of the CBD were in the 50 percent or more category

Table 11.—Number of trips per 100 dwelling units to the central business district in 1948 and 1955, classified according to mode of travel and distance of residence from the CBD

Distances from CBD	Automobile driver		Automobile passenger		Taxi passenger		Mass-transit passenger	
	1955	1948	1955	1948	1955	1948	1955	1948
0–2 miles	9.9	8.9	6.5	6.3	4.8	5.3	32.7	53.5
2–4 miles	15.2	17.8	9.7	11.2	3.0	3.4	29.8	52.3
4–6 miles	22.2	22.5	12.5	12.8	2.2	2.8	23.6	43.1
6–8 miles	19.0	18.2	11.8	10.9	.7	1.5	12.5	22.5
8–10 miles	21.9	21.7	14.6	13.1	.6	1.8	9.3	13.7
Average, total area	17.0	16.8	10.4	10.3	2.7	3.4	24.7	45.4

in 1948. By 1955 the use of the private automobile had increased to the point that even residents at this distance made less than 50 percent of their total "other" purpose trips by mass transit.

Modes of Travel

In 1948 and 1955 the proportion of to al residents trips to the CBD made by mass-transit passengers and taxi passengers decreased with distance from the CBD while those made by automobile drivers and automobile passengers increased (table 9). The major change which did take place between 1948 and 1955 was a shift in the mode of travel. In 1955 there was a smaller percentage of total residents trips to the CBD in each ring that were made by mass-transit passengers, whereas the proportion of CBD trips made by automobile drivers and automobile passengers increased.

In the subsequent discussion of taxi-passenger trips, it must be kept in mind that only residents taxi trips are included in this study. A very large proportion of taxi-passenger trips are made by nonresidents (transients and tourists).

Automobile drivers

The percentage distribution of trips to the CBD, classified according to mode of travel and distance, is shown in table 10 and figure 13. There was approximately the same percentage of automobile-driver trips to the CBD made by residents 2 to 4 and 4 to 6 miles from the CBD in 1948, 34.2 percent and 34.3 percent, respectively. This similarity disappeared in 1955. Residents within 4 to 6 miles made by far the largest percentage of automobile-driver trips to the CBD, 36.3 percent.

Although residents of the 2- to 4-mile ring had a substantially smaller average number of automobile-driver trips to the CBD per 100 dwelling units in 1948 than that observed for residents of the 4- to 6-mile ring (table 11), the 2- to 4-mile ring included more dwelling units. This offsetting factor accounted for the similar percentage of the total automobile-driver trips destined to the CBD by residents of these two rings in 1948.

In contrast, the 4- to 6-mile ring in 1955 contained approximately as many dwelling units as the 2- to 4-mile ring. This characteristic, plus the fact that in 1955 residents 4 to 6 miles from the CBD continued to make approximately the same number of automobile-driver trips to the CBD per 100 dwelling units as they did in 1948, while the number of trips per 100 dwelling units in the 2- to 4-mile ring decreased. These factors caused the change in the distribution pattern in 1955.

The general tendency in 1948 and 1955 was for the average number of automobile-driver trips to the CBD per 100 dwelling units to increase for the first few rings and then remain at about the same level beyond 6 miles of the CBD. With an increase in residential development in the outer two rings, the percentage distribution curve for automobile drivers might tend to flatten beyond 4 miles and possibly extend over a greater distance than in 1948. This trend appeared to be taking place in 1955, as residents 6 to 8 and 8 to 10 miles from the CBD made one-fourth (25.6 percent) of the total automobile-driver trips to the CBD. This compares with 17.4 percent in 1948.

The relationship between distance and the proportion of intra-area automobile-driver trips destined to the CBD remained the same during the period between 1948 and 1955; as distance from the CBD increased, the percentage of total intra-area automobile-driver trips destined to the CBD decreased (table 12). The percentage from each ring, however, was less in 1955 than in 1948.

The volume of automobile-driver trips to the CBD made by residents 6 to 8 miles from the CBD varied greatly for both years from those made by residents within 8 to 10 miles of the central area. Residents of both rings, however, made approximately the same percentage of their total intra-area automobile-driver trips to the CBD, 7.0 and 7.4 percent in 1948, and 5.1 and 5.0 percent in 1955.

With respect to the purposes for which residents made automobile-driver trips to the CBD, there was a general shift in the relative importance of work and "other" trips (table 8). The proportion of residents automobile-driver trips destined to the CBD for "other" purposes increased in each ring between 1948 and 1955, while the percentage for work decreased. By 1955, the percentage for "other" purposes had increased in the first two rings so that it was the principal purpose for residents within a 2-mile radius and ranked second for residents in the 2- to 4-mile ring. Work trips remained the dominant purpose for residents beyond 4 miles of the central area.

Automobile passengers

The distribution pattern of residents automobile-passenger trips to the CBD in 1948 and 1955 w as similar to that of automobile-driver trips to the CBD (table 10 and figure 13). A comparison of the change ratio for automobile-driver and automobile-passenger trips (table 12), however, shows that the proportion of residents automobile-passenger trips to the CBD changed less between 1948 and 1955 than did the proportion of automobile-driver trips.

The major difference in the purpose distribution was that automobile-passenger work trips had become even more common in 1955

Table 12.—Trips to the central business district in 1948 and 1955, classified according to mode of travel and distance of residence from the CBD

Mode of travel comparisons	0 to 2 miles		2 to 4 miles		4 to 6 miles		6 to 8 miles		8 to 10 miles		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Automobile driver trips:												
Total intra-area trips	96,933	65,207	243,930	191,562	402,803	232,776	308,297	110,909	106,292	28,326	1,158,255	628,780
Trips to CBD	10,431	7,960	20,978	19,315	29,859	19,345	15,787	7,763	5,285	2,098	82,340	56,481
Percentage of trips to CBD	10.8	12.2	8.6	10.1	7.4	8.3	5.1	7.0	5.0	7.4	7.1	9.0
1955 percentage/1948 percentage	.89		.85		.89		.73		.68		.79	
1955 trips to CBD/1948 trips to CBD	1.31		1.09		1.54		2.03		2.52		1.46	
Automobile passenger trips:												
Total intra-area trips	57,770	41,676	132,184	110,496	195,841	127,611	152,354	62,056	52,866	14,304	591,015	356,143
Trips to CBD	6,846	5,612	13,407	12,137	16,890	11,055	9,763	4,648	3,537	1,264	50,443	34,716
Percentage of trips to CBD	11.9	13.5	10.1	11.0	8.6	8.7	6.4	7.5	6.7	8.8	8.5	9.7
1955 percentage/1948 percentage	.88		.92		.99		.85		.76		.88	
1955 trips to CBD/1948 trips to CBD	1.22		1.10		1.53		2.10		2.80		1.45	
Taxi passenger trips:												
Total intra-area trips	20,986	17,653	18,624	16,888	11,100	8,873	5,983	2,416	1,403	646	58,096	46,476
Trips to CBD	5,018	4,704	4,140	3,692	2,988	2,387	570	619	155	171	12,871	11,573
Percentage of trips to CBD	23.9	26.6	22.2	21.9	26.9	26.9	9.5	25.6	11.0	26.5	22.2	24.9
1955 percentage/1948 percentage	.90		1.01		1.00		.37		.42		.89	
1955 trips to CBD/1948 trips to CBD	1.07		1.12		1.25		.92		.91		1.11	
Mass-transit passenger trips:												
Total intra-area trips	168,109	215,110	200,877	245,270	164,548	164,312	70,122	47,036	18,026	6,470	621,682	678,198
Trips to CBD	34,507	47,798	41,043	56,794	31,812	37,095	10,385	9,556	2,258	1,325	120,005	152,568
Percentage of trips to CBD	20.5	22.2	20.4	23.2	19.3	22.6	14.8	20.3	12.5	20.5	19.3	22.5
1955 percentage/1948 percentage	.92		.88		.85		.73		.61		.86	
1955 trips to CBD/1948 trips to CBD	.72		.72		.86		1.09		1.70		.79	

Table 13.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to purpose and sector of residence

Sectors	Work trips		Shopping trips		Other trips		All purposes	
	1955	1948	1955	1948	1955	1948	1955	1948
0	0.3	0.6	0.5	0.3	4.2	5.8	1.4	2.0
1	6.5	5.4	5.6	6.6	7.4	7.6	6.6	6.1
2	16.7	15.2	14.3	16.9	15.9	16.3	16.2	15.7
3	27.1	32.2	30.0	30.7	24.5	25.0	26.7	30.0
4	12.4	11.8	13.5	13.7	12.0	10.0	12.4	11.6
5	7.1	8.3	9.4	7.8	7.6	7.3	7.5	8.0
6	12.0	9.8	16.0	12.1	12.0	10.3	12.5	10.2
7	8.9	9.5	6.1	7.5	8.9	11.8	8.6	9.9
8	9.0	7.2	4.6	4.4	7.5	5.9	8.1	6.5
Total area	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

than they had been in 1948. Residents beyond 2 miles of the CBD made 63 percent or more of their total automobile-passenger trips to the CBD in 1955 for purposes of work. In the area beyond 6 miles, approximately 3 out of 4 residents traveling to the CBD as automobile passengers went there to work. In 1948 only residents of the 6- to 8-mile ring made over 60 percent of their automobile-passenger trips to the CBD for this purpose.

Taxi passengers

In comparison with other modes of travel, the percentage distribution of residents taxi-passenger trips made to the CBD changed only slightly between 1948 and 1955 (table 10 and figure 13). The largest percentage of reported taxi-passenger trips to the CBD were made by residents living closest to the CBD; those living within 4 miles made 72.5 percent of the total taxi trips to the CBD in 1948 and 71.2 percent in 1955.

The number of taxi-passenger trips to the CBD per 100 dwelling units was less in each distance ring in 1955 than it had been in 1948 (table 11), and in 1955 the number of trips per 100 dwelling units decreased with distance from the CBD. Because of this relationship, residential development beyond 4 miles of the CBD should have little effect on the percentage distribution of taxi-passenger trips to the CBD in the future.

Of significance was the change which occurred in the proportion of taxi-passenger trips made to the CBD by residents beyond 6 miles of the CBD (table 12). Although there were relatively few taxi-passenger trips made to the CBD by residents 6 to 8 and 8 to 10 miles from the central area, residents of both rings more than doubled the number of trips in the 7-year period. The percentage of total area trips going to the CBD, however, decreased. As indicated by the change ratio given in table 12, this decrease in percentage between 1948 and 1955 for the outer two rings was in sharp contrast to the percentage change for the first three rings.

Apparently the purposes for which taxi-passenger trips were made were less influenced by distance from the CBD than they were by other characteristics peculiar to the residents of the ring (table 8). The erratic nature of the percentages for the 8- to 10-mile ring is probably due to the small sample of taxi-passenger trips.

One trend in the purpose distribution was apparent in that the percentage of residents taxi-passenger trips to work increased in all but the 8- to 10-mile ring. By 1955, 50 percent or more of the taxi-passenger trips made to the CBD by residents within 8 miles of the CBD were for the purpose of work.

Mass-transit passengers

Of the four modes of travel studied, mass-transit passenger travel was the only mode that decreased in the overall volume of trips destined to the CBD. The residents accounting for this decrease lived within a 6-mile radius of the central area (table 12). Residents within this area still accounted for 92.8 percent of the mass-transit passenger trips destined to the CBD in 1948 and 89.5 percent in 1955 (table 10 and figure 13).

The decrease in the number of mass-transit passenger trips to the CBD per 100 dwelling units which occurred in each ring was equally significant (table 11). Since the number of trips per dwelling unit by automobile driver, automobile passenger, and taxi passenger changed only slightly, trips by mass transit accounted for the major proportion of the decrease within each ring. When residents changed their mode of travel to the CBD, it was usually a change from mass transit to one of the other modes.

In both years, as distance from the CBD increased, there was a progressive decrease in the number of mass-transit passenger trips per dwelling unit. This was in contrast to automobile-driver and automobile-passenger trips (table 11).

A change did occur in the relationship of distance and the percentage of residents intra-area mass-transit passenger trips destined to the CBD (table 12). In 1948, about the same proportion of CBD trips to total area trips was found for each 2-mile ring. By 1955, however, the proportion of residents mass-transit passenger trips destined to the CBD was progressively smaller with increased distance.

Work trips in both years accounted for the major proportion of mass-transit passenger trips to the CBD (table 8), and the proportion increased for each ring between 1948 and 1955. In contrast, the proportion of mass-transit shopping trips to the CBD decreased for each ring except for the area closest to the CBD.

ANALYSIS OF DIRECTION FACTOR

The sector subdivisions used in this study are much like a watershed of a stream. In general, along these sector divisions flow the major inbound and outbound CBD traffic. As a result, the radial patterns of traffic provide still another means, besides distance, in analyzing the characteristics of travel to the CBD.

In the study of sectors, a different approach is taken from that used in the study of rings. The variation in distances from the CBD was the major criterion for interpreting much of the information in the ring analysis. In the study of sectors, the radial growth and distribution of population, socioeconomic characteristics, and historical trends come into play in influencing the patterns that develop and the changes which take place. It has been said that "* * * the different types of residential areas tend to grow outward along rather distinct radii, and new growth on the arc of a given sector tends to take on the character of the initial growth in that sector * * *." [5]

Although there was a shifting between sectors, there were no major changes in the overall pattern of residents CBD trip distribution by sectors as there had been by 2-mile rings (table 13). The percentage of trips to the CBD from 7 of the 9 sectors varied no more than ± 1.6 percentage points from what it had been in 1948, and even though the percentage of CBD trips made by residents of sector 3 decreased slightly due to the relative increases in the other sectors, the major peak in residents trip distribution remained in sector 3.

This pattern of distribution and the relative changes which took place between the two years are very similar to that which occurred for population and dwelling units, and it suggests a close relationship between the distribution of residents trips to the CBD and the distribution of residential development (fig. 14). It did not necessarily follow, however, that residents of a sector with a large number of trips to the CBD per dwelling unit also made a large percentage of all residents CBD trips (tables 13 and 14). It was not only the number of trips per dwelling unit that affected the pattern of distribution, but also the amount of residential development.

A change did appear to be taking place in the travel pattern of certain groups of contiguous sectors, which in 1948 had practically the same percentage of total trips destined to the CBD (table 15). In 1948 the percentage of CBD trips made by residents of sectors 1, 2, and 3 ranged from 16.0 to 16.4 percent; that by residents of sectors 4, 5, and 6 ranged from 13.5 to 14.3 percent; and that by residents of sectors 7 and 8 ranged from 12.3 to 12.5 percent. By 1955, all the percentages had decreased, and the decrease was such that the range between percentages for these sector groups was somewhat larger than in 1948.

It is difficult to say whether these differences between the 1948 and 1955 pattern were due to a trend that was taking place or to chance. If, however, these differences persist or increase, they may indicate a breaking up of the groups and the beginning of new sector groupings with a similar orientation of intra-area trips destined to the CBD.

[5] *The Structure and Growth of Residential Neighborhoods in American Cities.* Federal Housing Administration, Washington, D.C., 1939, p. 114.

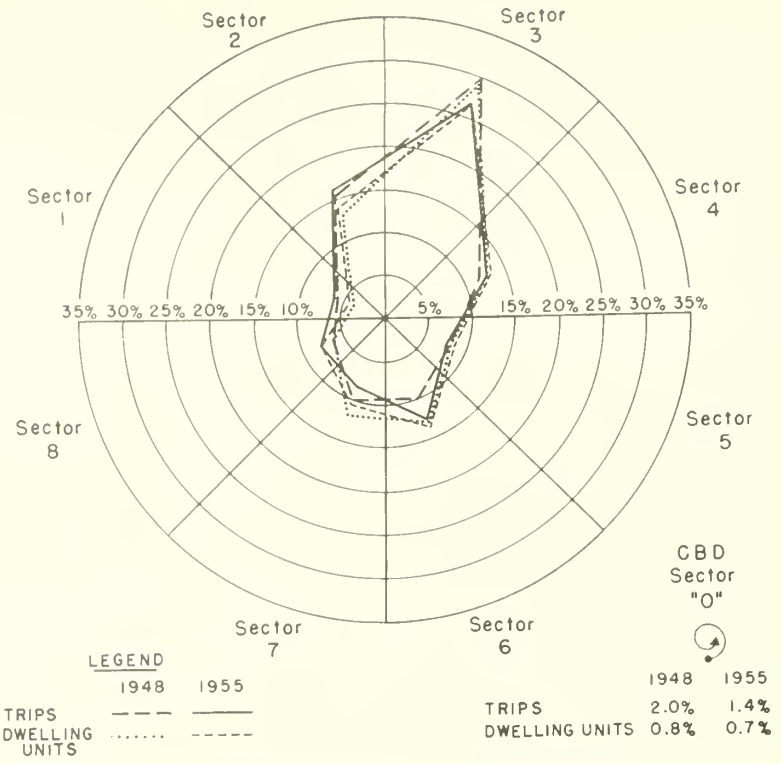

Figure 14.—Percentage distribution of residents CBD trips and dwelling units, according to sector of residence.

Trip Purposes

The change in the percentage distribution of CBD trip purposes varied from sector to sector (table 16). The overall trend, however, was one of decreasing differences between sectors in the proportion of residents trips for work and "other" purposes. Excluding trips made by residents of sector zero, the range between the highest and the lowest percentage of CBD work trips decreased from 14.4 percent in 1948 to 10.6 percent in 1955. More striking was the decrease in the range for "other" trips, 11.1 to 5.4 percent.

Shopping trips, on the other hand, increased slightly in the percentage range among sectors. The spread among the percentages in 1948 was 6.9 percent, and in 1955 it was 8.4 percent. This characteristic of shopping trips can be attributed to the unbalanced development of commercial facilities among the sectors. An example of this is indicated by the distribution of trip purposes for residents of sectors 5 and 6. The area included by these sectors had a relatively sparse development of major commercial activity in relation to residential growth prior to the 1948 study, and comparatively few shopping concentrations had been added by 1955. The result, as indicated in table 16, was that residents of these sectors maintained a higher percentage of their trips to the CBD for purposes of shopping than did the residents of the other sectors.

Work trips

The 1955 distribution pattern among the sectors of residents work trips to the CBD varied no more than ±2.2 percentage points in all sectors, except sector 3, from the corresponding percentage in 1948 (table 13 and figure 15). Although the percentage made by residents of sector 3 decreased, the large number of residents of this sector still accounted for the major peak in the pattern of work trip distribution (32.2 percent in 1948 and 27.1 percent in 1955).

Figure 16 shows the relationship between the number of CBD work trips per 100 dwelling units of a sector in 1948 and 1955 and the number of employment opportunities per 100 dwelling units in that sector. With some exceptions, the general trend in each of the years was for the number of CBD work trips per 100 dwelling units to decrease as the number of employment opportunities per 100 dwelling units in a given sector increased. Employees tend to work at locations closer to their place of residence if there is an opportunity to do so, or conversely, they tend to live in the vicinity of their work.

In comparing the positions of the plotted symbols in figure 16 for the two years, it is seen that for a given sector the plotted symbols for 1955 fall to the left of those for 1948, except in the case of sector 7. The reason for this relation is that the rate of residential development in each of the sectors except sector 7 exceeded the rate of growth in employment opportunities in the corresponding sector.

The percentage of residents intra-area work trips destined to the CBD decreased in all sectors with the exception of sector 1 (table 15). The amount of decrease varied with each sector, resulting in a new geographic relationship between sectors 1 through 5. Beginning with sector 1 and following around the central area through sector 5, it is apparent that there was a continual decrease between sectors in the proportion of 1955 intra-area work trips destined to the CBD. Although there are exceptions, this transition appears to be one from sectors with high average family incomes and with a large proportion of professional and white-collar employees to one of lower average family incomes and a large proportion of blue-collar employees. Thus, there is an indirect inference that an increasing number of employment opportunities were afforded blue-collar employees outside the CBD in 1955.

The increase in automobile ownership caused a decline in the proportion of mass-transit work trips made by sector residents (tables 17 and 18). For example, in both 1948 and 1955 the average number of automobiles owned per 100 dwelling units by residents of sectors 0, 3, 5, and 6 was below the study area average. On the other hand, the percentage of work trips by mass transit for residents of each of these sectors was well above the average. As the number of automobiles owned per dwelling unit increased between 1948 and 1955 in all sectors, except sector zero, there was a decrease, with the exception of sectors 0 and 1, in the proportion of work trips by mass transit and a corresponding increase in those by automobile.

Shopping trips

The distribution pattern of sector residents shopping trips to the CBD in 1955 did not change materially from that in 1948 (table 13 and figure 15); six of the sectors had a change within ±1.5 percentage points. The largest relative change, an approximate 4-percent increase, took place in sector 6, and was caused by a population increase of approximately 53,000 persons, the second largest increase in the study area. The residents of sector 6 were not served as adequately by major suburban shopping facilities as were other sectors. With few new commercial developments in this sector between 1948 and 1955, and with the large increase in population, there was an increase in the number of CBD trips.

Table 14.—Number of trips per 100 dwelling units to the central business district in 1948 and 1955, classified according to purpose and sector of residence

Sectors	Work trips		Shopping trips		Other trips		All purposes	
	1955	1948	1955	1948	1955	1948	1955	1948
0	15.4	32.9	4.9	4.0	91.9	148.0	112.2	184.9
1	47.5	61.1	7.8	17.3	24.1	40.3	79.4	118.8
2	40.2	51.3	6.7	13.0	17.1	25.6	64.1	89.9
3	34.2	49.4	7.3	10.7	13.8	17.8	55.3	77.9
4	31.5	41.6	6.6	11.0	13.6	16.3	51.8	68.9
5	27.3	45.3	7.0	9.6	13.1	18.3	47.4	73.3
6	29.3	33.4	7.5	9.4	13.1	16.3	49.9	59.1
7	27.7	35.8	3.7	6.5	12.4	20.7	43.8	62.9
8	36.7	48.9	3.6	6.7	13.6	18.6	53.9	74.3
Average, total area	33.4	44.9	6.4	10.2	14.9	20.8	54.8	76.0

Table 15.—Trips to the central business district in 1948 and 1955, classified according to purpose and sector of residence

Trip purpose comparisons	Sector 0		Sector 1		Sector 2		Sector 3		Sector 4		Sector 5		Sector 6		Sector 7		Sector 8		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Work trips:																				
Total intra-area trips	1,662	2,127	27,354	21,940	73,913	54,815	150,090	135,349	85,549	66,587	51,641	40,752	72,810	51,684	58,006	45,624	54,358	33,316	575,383	452,194
Trips to CBD	511	911	10,581	8,071	27,075	22,952	43,895	48,609	20,067	17,842	11,426	12,560	19,520	14,744	14,393	14,310	14,624	10,867	162,092	150,866
Percentage of trips to the CBD	30.7	42.8	38.7	36.8	36.6	41.9	29.2	35.9	23.5	26.8	21.1	30.8	26.8	28.5	24.8	31.4	26.9	32.6	26.2	33.4
1955 percentage/1948 percentage	.72	-------	1.05	-------	.87	-------	.81	-------	.88	-------	.72	-------	.94	-------	.79	-------	.83	-------	.84	----------
1955 trips to CBD/1948 trips to CBD	.56	-------	1.31	-------	1.18	-------	.90	-------	1.12	-------	.91	-------	1.32	-------	1.01	-------	1.35	-------	1.07	----------
Shopping trips:																				
Total intra-area trips	544	156	12,242	7,287	31,426	20,334	36,848	24,597	23,936	13,302	12,852	7,631	19,675	10,242	25,213	17,025	29,481	9,586	192,217	110,160
Trips to CBD	163	112	1,745	2,289	4,476	5,816	9,354	10,570	4,223	4,723	2,947	2,665	4,995	4,152	1,913	2,587	1,419	1,497	31,235	34,411
Percentage of trips to the CBD	30.0	71.8	14.3	31.4	14.2	28.6	25.4	43.0	17.6	35.5	22.9	34.9	25.4	40.5	7.6	15.2	4.8	15.6	16.2	31.2
1955 percentage/1948 percentage	.42	-------	.46	-------	.50	-------	.59	-------	.50	-------	.66	-------	.63	-------	.50	-------	.31	-------	.52	----------
1955 trips to CBD/1948 trips to CBD	1.46	-------	.76	-------	.77	-------	.88	-------	.89	-------	1.11	-------	1.20	-------	.74	-------	.95	-------	.91	----------
Other trips:																				
Total intra-area trips	4,172	5,490	93,861	66,189	255,963	176,614	377,695	314,107	238,828	139,503	131,616	93,144	197,096	122,839	178,191	138,605	184,026	90,752	1,661,448	1,147,243
Trips to CBD	3,054	4,094	5,365	5,321	11,531	11,427	17,691	17,524	8,683	7,009	5,463	5,083	8,704	7,187	6,429	8,287	5,412	4,129	72,332	70,061
Percentage of trips to the CBD	73.2	74.6	5.7	8.0	4.5	6.5	4.7	5.6	3.6	5.0	4.2	5.5	4.4	5.9	3.6	6.0	2.9	4.5	4.4	6.1
1955 percentage/1948 percentage	.98	-------	.71	-------	.69	-------	.84	-------	.72	-------	.75	-------	.75	-------	.60	-------	.64	-------	.72	----------
1955 trips to CBD/1948 trips to CBD	.75	-------	1.01	-------	1.01	-------	1.01	-------	1.24	-------	1.07	-------	1.21	-------	.78	-------	1.31	-------	1.03	----------
All purposes:																				
Total intra-area trips	6,378	7,773	133,457	95,416	361,302	251,763	564,633	474,053	348,313	219,392	196,109	141,527	289,581	184,765	261,410	201,254	267,865	133,654	2,429,048	1,709,597
Trips to CBD	3,728	5,117	17,691	15,681	43,082	40,195	70,940	76,703	32,973	29,574	19,836	20,308	33,219	26,083	22,735	25,184	21,455	16,493	265,659	255,338
Percentage of trips to the CBD	58.5	65.8	13.3	16.4	11.9	16.0	12.6	16.2	9.5	13.5	10.1	14.3	11.5	14.1	8.7	12.5	8.0	12.3	10.9	14.9
1955 percentage/1948 percentage	.89	-------	.81	-------	.74	-------	.78	-------	.70	-------	.71	-------	.82	-------	.70	-------	.65	-------	.73	----------
1955 trips to CBD/1948 trips to CBD	.73	-------	1.13	-------	1.07	-------	.92	-------	1.11	-------	.98	-------	1.27	-------	.90	-------	1.30	-------	1.04	----------

Table 16.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to trip purpose of mode of travel and sector of residence

Trip purpose of mode of travel	Sector 0		Sector 1		Sector 2		Sector 3		Sector 4		Sector 5		Sector 6		Sector 7		Sector 8		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Automobile-driver trips to—																				
Work	-----	14.2	54.5	49.1	55.3	55.3	50.5	62.5	52.4	60.1	44.6	54.3	44.4	56.7	53.1	46.7	56.0	59.2	51.2	55.9
Ratio, 1955/1948	-----	-----	1.11	-----	1.00	-----	.81	-----	.87	-----	.82	-----	.78	-----	1.14	-----	.95	-----	.92	-----
Shop	-----	-----	5.9	8.0	5.9	6.2	6.2	6.1	8.3	7.0	12.1	7.5	10.1	7.5	7.5	8.7	4.2	5.1	7.2	6.8
Ratio, 1955/1948	-----	-----	.74	-----	.95	-----	1.02	-----	1.19	-----	1.61	-----	1.35	-----	.86	-----	.82	-----	1.06	-----
Other	100.0	85.8	39.6	42.9	38.8	38.5	43.3	31.4	39.3	32.9	43.3	38.2	45.5	35.8	39.4	44.6	39.8	35.7	41.6	37.3
Ratio, 1955/1948	1.17	-----	.92	-----	1.01	-----	1.38	-----	1.20	-----	1.13	-----	1.27	-----	.88	-----	1.12	-----	1.12	-----
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Automobile-passenger trips to—																				
Work	-----	17.2	44.7	51.6	63.3	58.6	71.3	57.2	72.8	52.0	72.7	57.1	63.0	51.4	62.9	51.4	73.2	66.2	66.2	55.1
Ratio, 1955/1948	-----	-----	.87	-----	1.08	-----	1.25	-----	1.40	-----	1.27	-----	1.23	-----	1.22	-----	1.11	-----	1.20	-----
Shop	8.8	-----	9.2	11.4	10.5	9.9	8.1	10.3	6.8	14.6	10.4	13.7	14.2	11.1	7.6	8.2	5.1	6.0	8.9	10.3
Ratio, 1955/1948	-----	-----	.81	-----	1.06	-----	.79	-----	.47	-----	.76	-----	1.28	-----	.93	-----	.85	-----	.86	-----
Other	91.2	82.8	46.1	37.0	26.2	31.5	20.6	32.5	20.4	33.4	16.9	29.2	22.8	37.5	29.5	40.4	21.7	27.8	24.9	34.6
Ratio, 1955/1948	1.10	-----	1.25	-----	.83	-----	.63	-----	.61	-----	.58	-----	.61	-----	.73	-----	.78	-----	.72	-----
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Taxi-passenger trips to —																				
Work	21.3	18.1	60.2	41.5	60.8	48.8	58.3	59.8	36.6	57.1	64.2	38.3	57.9	17.0	22.0	41.7	56.3	60.0	54.7	48.8
Ratio, 1955/1948	1.18	-----	1.45	-----	1.25	-----	.98	-----	.64	-----	1.68	-----	3.41	-----	.53	-----	.94	-----	1.12	-----
Shop	7.4	-----	9.6	15.2	4.7	12.6	15.6	8.4	16.9	28.6	17.9	26.0	19.6	10.3	24.6	-----	12.5	-----	12.3	11.4
Ratio, 1955/1948	-----	-----	.63	-----	.37	-----	1.86	-----	.59	-----	.69	-----	1.90	-----	-----	-----	-----	-----	1.08	-----
Other	71.3	81.9	30.2	43.3	34.5	38.6	26.1	31.8	46.5	14.3	17.9	35.7	22.5	72.7	53.4	58.3	31.2	40.0	33.0	39.8
Ratio, 1955/1948	.87	-----	.70	-----	.89	-----	.82	-----	3.25	-----	.50	-----	.31	-----	.92	-----	.78	-----	.83	-----
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Mass-transit passenger trips to—																				
Work	17.7	18.5	69.9	54.5	68.9	58.8	64.7	64.6	63.0	62.7	60.4	66.3	66.3	59.0	77.6	65.4	81.6	72.1	66.2	62.0
Ratio, 1955/1948	.96	-----	1.28	-----	1.17	-----	1.00	-----	1.01	-----	.91	-----	1.12	-----	1.19	-----	1.13	-----	1.07	-----
Shop	3.4	3.2	13.0	20.3	15.0	19.6	17.7	16.4	19.0	19.4	17.8	13.7	18.2	20.1	9.0	12.6	10.8	14.8	16.0	16.8
Ratio, 1955/1948	1.06	-----	.64	-----	.77	-----	1.08	-----	.98	-----	1.30	-----	.91	-----	.71	-----	.73	-----	.95	-----
Other	78.9	78.3	17.1	25.2	16.1	21.6	17.6	19.0	18.0	17.9	21.8	20.0	15.5	20.9	13.4	22.0	7.6	13.1	17.8	21.2
Ratio, 1955/1948	1.01	-----	.68	-----	.75	-----	.93	-----	1.01	-----	1.09	-----	.74	-----	.61	-----	.58	-----	.84	-----
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
All modes of travel to—																				
Work	13.7	17.8	59.8	51.5	62.8	57.1	61.9	63.4	60.9	60.3	57.6	61.9	58.8	56.5	63.3	56.8	68.2	65.9	61.0	59.1
Ratio, 1955/1948	.77	-----	1.16	-----	1.10	-----	.98	-----	1.01	-----	.93	-----	1.04	-----	1.11	-----	1.01	-----	1.03	-----
Shop	4.4	2.2	9.9	14.6	10.4	14.5	13.2	13.8	12.8	16.0	14.9	13.1	15.0	15.9	8.4	10.3	6.6	9.1	11.8	13.5
Ratio, 1955/1948	2.00	-----	.68	-----	.72	-----	.96	-----	.80	-----	1.14	-----	.94	-----	.82	-----	.73	-----	.87	-----
Other	81.9	80.0	30.3	33.9	26.8	28.4	24.9	22.8	26.3	23.7	27.5	25.0	26.2	27.6	28.3	32.9	25.2	25.0	27.2	27.4
Ratio, 1955/1948	1.02	-----	.89	-----	.94	-----	1.09	-----	1.11	-----	1.10	-----	.95	-----	.86	-----	1.01	-----	.99	-----
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

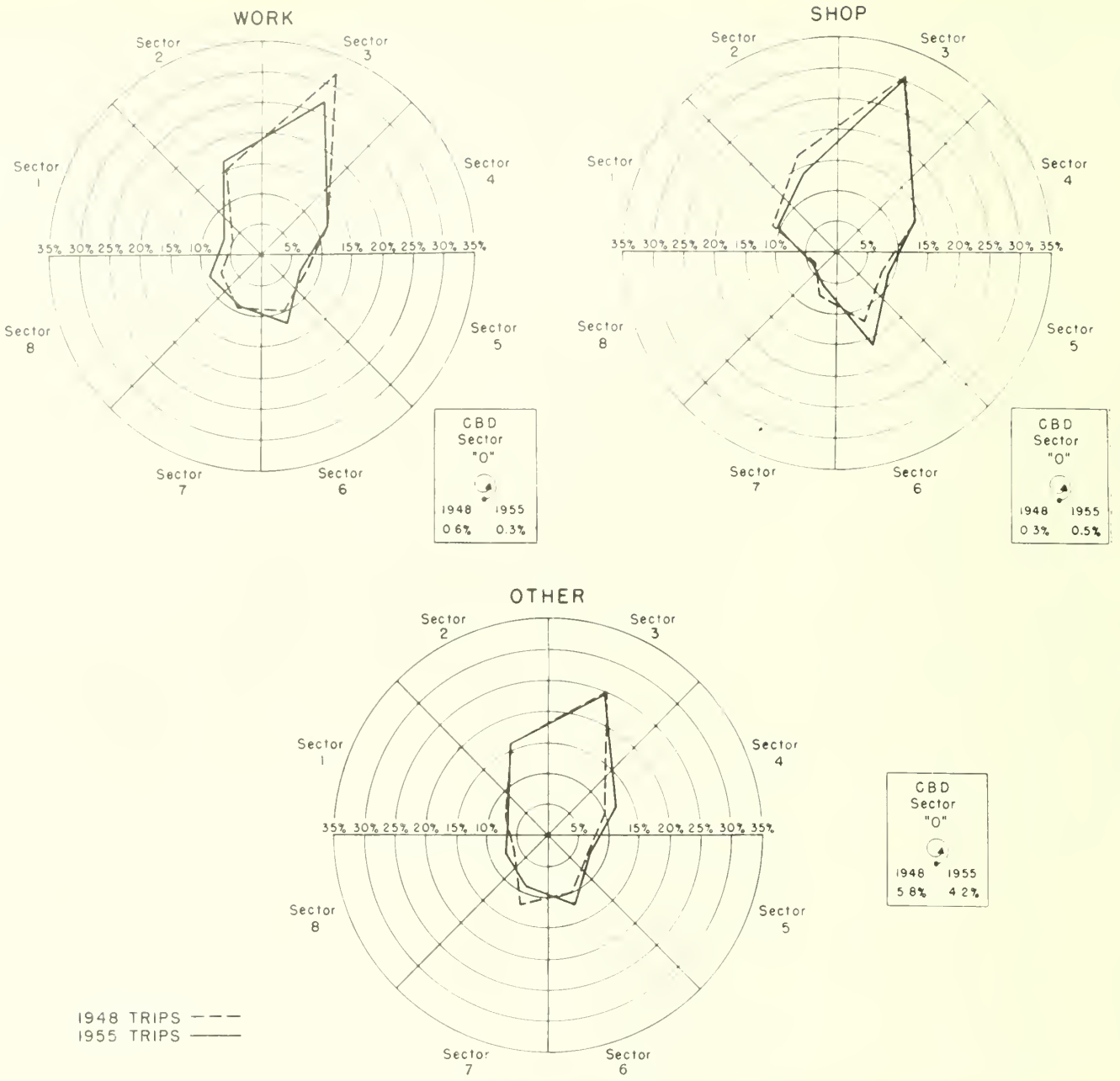

Figure 15.—Percentage distribution of work, shop, and other purpose trips to the CBD, according to sector of residence.

The number of commercial concentrations increased in the various sectors, and thus contributed in part to the decrease in the number of shopping trips per dwelling unit. The increasing uniformity of CBD shopping trips by sector residents illustrates, however, that the central area continued to offer certain shopping advantages that were not satisfied elsewhere (table 14). If residents trips of sectors 0, 7, and 8 are excluded, the range between the sector with the greatest number of shopping trips per dwelling unit and the sector with the least number was only 1.2 trips. In 1948 the range was 7.9 trips.

Although the actual volume of intra-area shopping trips increased between 1948 and 1955, it is apparent from the change ratio (table 15) that the orientation of these trips by residents of each sector had changed significantly. The percentage of intra-area shopping trips to the CBD in 1955 decreased by one-third for residents in all sectors, and for residents in six of the sectors the percentage decreased by one-half or more.

Shopping trips for the two years by residents of sector 8 illustrate an extreme of this decreasing percentage of intra-area trips to the CBD. This sector is relatively more suburban in character than many of the other sectors at similar distances from the CBD. In 1948, 1,497 or 15.6 percent of 9,586 shopping trips by residents of sector 8 were destined to the CBD. In 1955 intra-area shopping trips had increased to 29,481, but the volume to the CBD remained approximately the same as in 1948 (1,419 trips), a drop from 15.6 to 4.8 percent.

There are certain geographic factors which had a great effect on trips to the CBD by residents of Virginia, especially shopping trips. It would be well, therefore, to discuss more fully sectors 7 and 8 which make up the Virginia portion of the study area. Although 2 of the 4 bridges across the Potomac River lead directly to the CBD, there was a low orientation in the 1948 study of shopping trips to the CBD by residents of sectors 7 and 8 compared with similar trips made by residents of the other sectors (table 15). The relatively low percentages of shopping trips to the CBD became more pronounced in 1955 as increased suburban commercial development took place.

This recent history of shopping trips by residents of sectors 7 and 8 illustrates a lesser dependency on the Washington CBD for shopping purposes than was the case for other sectors. The Potomac River, because of its restricting effect on direct travel, is a psychological as well as a physical barrier to travel by Virginians to the Washington CBD. As a result, several large commercial concentrations, such as that found in the Clarendon section of Arlington, had developed before the 1948 study, and they were serving many of the retail needs of Virginia residents. Since that time, these centers have been greatly augmented by other regional as well as neighborhood

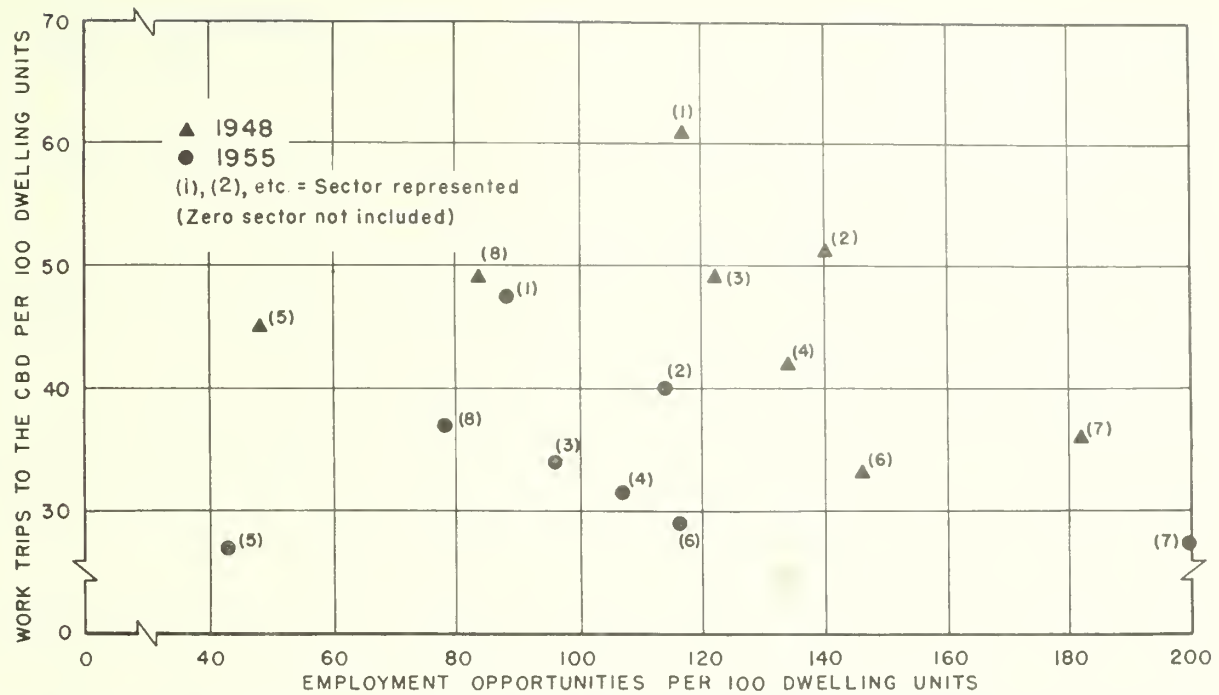

Figure 16.—Relation of the number of CBD work trips per 100 dwelling units to the number of employment opportunities per 100 dwelling units, according to sector of residence.

type retail concentrations. Thus, there appeared to be an even lower orientation of shopping trips to the CBD in 1955.

The close tie that is usually considered to exist between shopping trips to the CBD and mass transit is largely substantiated by the data for 1948 (table 18). In 1948, residents of each of the 9 sectors made 60 percent or more of their CBD shopping trips by mass transit. This relation had changed by 1955 when only sectors 3, 4, and 5 maintained as high a percentage. Even in these three sectors, the percentage was less in 1955 than it had been in 1948.

The variation between sectors in the percentage change in mode of travel might best be explained by comparing the CBD shopping trips made by residents of sectors 3 and 7. Sector 3 had convenient mass-transit service, and there was a below area average increase in automobiles owned per 100 dwelling units (24.0 percent). Between 1948 and 1955, the percentage of CBD shopping trips by mass transit decreased from 82.9 to 72.0 percent in sector 3. Even with this decrease, the percentage remained high. In contrast, trips made by residents of sector 7 changed radically. Sector 7 had a relatively less convenient mass-transit system than sector 3, but there was a very large increase in the number of automobiles per 100 dwelling units (54.0 percent). In 1948 mass transit carried nearly two-thirds (62.1 percent) of all the residents of sector 7 destined to the CBD for shopping. By 1955 the proportion was slightly more than one-third (37.5 percent).

Other purpose trips

As mentioned previously in the study of distance rings, it is possible only to make generalizations of the changes that took place in the "other" trip purpose category. This group, however, did show a similarity in several characteristics that were found for work and shopping trips.

The pattern of "other" trip purpose distribution changed only slightly between 1948 and 1955. Trips by residents of 4 of the 9 sectors in 1955 were within ±0.5 percent of the 1948 percentage, and 3 other sectors were within ±1.7 percentage points (table 13 and figure 15). As in work and shopping trips, the peak trip distribution was in sector 3 which had the largest number of residents.

The number of "other" purpose trips to the CBD per 100 dwelling units decreased for residents of all sectors (table 14), and the trend was toward greater uniformity in the distribution of trips per dwelling unit between each of the sectors, particularly sectors 3 through 8.

The distribution of the modes of travel by which "other" purpose trips were made indicates a large increase in the percentage of trips by automobile drivers for each sector between 1948 and 1955 (table 18). Here again, this change in mode of travel was apparently due to an increase in the automobile ownership ratio. In 1948 residents of only two sectors, 1 and 8, made 40 percent or more of their CBD "other" purpose trips as automobile drivers. At that time, these two sectors contained the largest number of automobiles per dwelling unit. With an increase in the number of automobiles owned, residents of each sector in 1955, except sector zero, made 40 percent or more of their "other" purpose trips as automobile drivers.

Modes of Travel

In 1948 and 1955, there was an inverse relationship between automobile ownership and the percentage of residents CBD trips made by mass-transit passengers. If the sectors are listed in accordance with automobiles owned per 100 dwelling units from the highest to the lowest and then by the percentage of residents CBD trips made by mass-transit passengers (table 18) from the lowest to the highest, it will be seen that for both years the sequence of the sectors was nearly the same. Exceptions to this sequence were sectors 4 and 7 in 1948 and 1, 2, and 7 in 1955.

Since there was a general tendency for smaller percentages of mass-transit passenger trips to be made by residents of sectors with a greater number of automobiles owned per dwelling unit, it followed that with increased automobile ownership within each sector, with the exception of sector zero, the percentage of CBD trips by mass transit decreased. The changes which occurred within each sector in the percentage distribution of mode of travel to the CBD resulted in an increasing similarity of these percentages between sectors (table 18).

To illustrate this increasing uniformity, residents of sectors 1 through 6 (District of

Table 17.—Average annual family income for 1955 and average number of automobiles owned per 100 dwelling units, grouped by sector of residence

Sectors	Average income, 1955 [1]	Number of automobiles owned per 100 dwelling units		
		1955	1948	Ratio, 1955/1948
0	$4,300	19	23	0.83
1	10,100	96	86	1.12
2	9,900	82	70	1.17
3	5,600	62	50	1.24
4	5,900	88	71	1.24
5	4,800	70	52	1.35
6	5,400	73	53	1.38
7	7,200	91	59	1.54
8	8,300	109	91	1.20
Average, total area	6,700	79	61	1.30

[1] Figures for 1948 are not shown because the data for the 2 years are not comparable.

Table 18.—Percentage distribution of trips to the central business district in 1948 and 1955, classified according to mode of travel of trip purpose and by sector of residence

Mode of travel of trip purpose	Sector 0		Sector 1		Sector 2		Sector 3		Sector 4		Sector 5		Sector 6		Sector 7		Sector 8		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Work trips by—																				
Automobile drivers	-------	9.8	28.2	30.2	28.0	22.8	20.4	15.6	28.5	22.9	23.8	16.0	23.0	22.2	33.2	23.1	33.7	32.4	26.0	20.9
Ratio, 1955/1948	-------	-------	.93	-------	1.23	-------	1.31	-------	1.25	-------	1.49	-------	1.04	-------	1.44	-------	1.04	-------	1.24	-------
Automobile passengers	-------	12.5	13.5	17.3	18.9	14.9	18.5	8.9	25.6	13.4	20.0	10.4	19.5	11.7	23.1	16.0	28.9	19.9	20.6	12.7
Ratio, 1955/1948	-------	-------	.78	-------	1.27	-------	2.08	-------	1.91	-------	1.92	-------	1.67	-------	1.44	-------	1.45	-------	1.62	-------
Taxi passengers	24.1	5.2	8.2	4.8	8.0	7.0	5.0	4.4	1.7	1.9	2.3	2.7	3.4	.7	.7	2.7	2.2	3.0	4.4	3.7
Ratio, 1955/1948	4.63	-------	1.71	-------	1.14	-------	1.14	-------	.90	-------	.85	-------	4.86	-------	.26	-------	.73	-------	1.19	-------
Mass-transit passengers	75.9	72.5	50.1	47.7	45.1	55.3	56.1	71.1	44.2	61.8	53.9	70.9	54.1	65.4	43.0	58.2	35.2	44.7	49.0	62.7
Ratio, 1955/1948	1.05	-------	1.05	-------	.82	-------	.79	-------	.72	-------	.76	-------	.83	-------	.74	-------	.79	-------	.78	-------
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Shopping trips by—																				
Automobile drivers	-------	-------	18.6	17.5	18.0	10.1	11.9	7.0	21.4	10.1	24.9	10.3	20.4	10.4	35.2	23.7	26.3	20.3	19.0	11.1
Ratio, 1955/1948	-------	-------	1.06	-------	1.78	-------	1.70	-------	2.12	-------	2.42	-------	1.96	-------	1.49	-------	1.30	-------	1.71	-------
Automobile passengers	28.2	-------	16.9	13.5	18.9	9.9	9.8	7.3	11.3	14.3	11.1	11.8	17.2	9.0	21.1	14.2	20.7	13.0	14.3	10.4
Ratio, 1955/1948	-------	-------	1.25	-------	1.91	-------	1.34	-------	.79	-------	.94	-------	1.91	-------	1.49	-------	1.59	-------	1.38	-------
Taxi passengers	26.4	-------	7.9	6.2	3.8	7.2	6.3	2.8	3.7	3.5	2.4	8.7	4.5	1.5	6.2	-------	5.1	-------	5.1	3.9
Ratio, 1955/1948	-------	-------	1.27	-------	.53	-------	2.25	-------	1.06	-------	.28	-------	3.00	-------	-------	-------	-------	-------	1.31	-------
Mass-transit passengers	45.4	100.0	56.6	62.8	59.3	72.8	72.0	82.9	63.6	72.1	61.6	69.2	57.9	79.1	37.5	62.1	47.9	66.7	61.6	74.6
Ratio, 1955/1948	.45	-------	.90	-------	.82	-------	.87	-------	.88	-------	.89	-------	.73	-------	.60	-------	.72	-------	.83	-------
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
Other trips by																				
Automobile drivers	14.5	13.1	40.5	40.0	46.2	31.8	43.5	21.8	49.1	31.9	48.3	27.8	53.0	28.7	55.1	38.1	64.6	51.4	47.3	30.1
Ratio, 1955/1948	1.10	-------	1.01	-------	1.45	-------	2.00	-------	1.55	-------	1.74	-------	1.85	-------	1.45	-------	1.26	-------	1.57	-------
Automobile passengers	15.5	13.4	27.3	18.8	18.4	16.1	13.2	14.0	16.6	21.9	9.7	13.2	15.8	17.5	24.2	21.7	23.2	22.0	17.3	17.1
Ratio, 1955/1948	1.16	-------	1.45	-------	1.14	-------	.94	-------	.76	-------	.74	-------	.90	-------	1.12	-------	1.06	-------	1.01	-------
Taxi passengers	13.5	5.2	8.1	7.6	10.6	11.2	5.6	6.4	4.9	1.2	1.3	6.2	2.9	6.1	4.0	6.4	3.3	5.2	5.9	6.6
Ratio, 1955/1948	2.60	-------	1.07	-------	.95	-------	.88	-------	4.08	-------	.21	-------	.48	-------	.63	-------	.64	-------	.89	-------
Mass-transit passengers	56.5	68.3	24.1	33.6	24.8	40.9	37.7	57.8	29.1	45.0	40.7	52.8	28.3	47.7	16.7	33.8	8.9	21.4	29.5	46.2
Ratio, 1955/1948	.83	-------	.72	-------	.61	-------	.65	-------	.65	-------	.77	-------	.59	-------	.49	-------	.42	-------	.64	-------
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
All purposes by—																				
Automobile drivers	11.9	12.3	31.0	31.7	31.9	23.5	25.1	15.8	33.1	23.0	30.7	18.2	30.5	22.1	39.6	28.1	41.0	36.1	31.0	22.1
Ratio, 1955/1948	.97	-------	.98	-------	1.36	-------	1.59	-------	1.44	-------	1.69	-------	1.38	-------	1.41	-------	1.14	-------	1.40	-------
Automobile passengers	13.9	12.9	18.0	17.2	18.7	14.5	16.0	9.8	21.4	15.5	15.9	11.3	18.2	12.9	23.2	17.7	26.9	19.8	19.0	13.6
Ratio, 1955/1948	1.08	-------	1.05	-------	1.29	-------	1.63	-------	1.38	-------	1.41	-------	1.41	-------	1.31	-------	1.36	-------	1.40	-------
Taxi passengers	15.5	5.1	8.1	6.0	8.3	8.3	5.3	4.7	2.8	2.0	2.0	4.4	3.4	2.3	2.1	3.6	2.7	3.2	4.8	4.5
Ratio, 1955/1948	3.04	-------	1.35	-------	1.00	-------	1.13	-------	1.40	-------	.46	-------	1.48	-------	.58	-------	.84	-------	1.07	-------
Mass-transit passengers	58.7	69.7	42.9	45.1	41.1	53.7	53.6	69.7	42.7	59.5	51.4	66.1	47.9	62.7	35.1	50.6	29.4	40.9	45.2	59.8
Ratio, 1955/1948	.84	-------	.95	-------	.77	-------	.77	-------	.72	-------	.78	-------	.76	-------	.69	-------	.72	-------	.76	-------
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Table 19.—Trips to the central business district in 1948 and 1955, classified according to mode of travel and sector of residence

Mode of travel comparisons	Sector 0		Sector 1		Sector 2		Sector 3		Sector 4		Sector 5		Sector 6		Sector 7		Sector 8		Total area	
	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948	1955	1948
Automobile driver trips:																				
Total intra-area trips	1,141	1,032	65,152	40,488	186,146	105,257	226,023	136,638	172,247	83,868	82,702	39,873	131,493	64,564	143,757	89,014	149,594	68,046	1,158,255	628,780
Trips to CBD	442	628	5,483	4,965	13,725	9,459	17,776	12,138	10,919	6,797	6,089	3,704	10,124	5,769	8,994	7,072	8,788	5,949	82,340	56,481
Percentage of trips to the CBD	38.7	60.9	8.4	12.3	7.4	9.0	7.9	8.9	6.3	8.1	7.4	9.3	7.7	8.9	6.3	7.9	5.9	8.7	7.1	9.0
1955 percentage/1948 percentage	.64	-------	.68	-------	.82	-------	.89	-------	.78	-------	.80	-------	.87	-------	.80	-------	.68	-------	.79	-------
1955 trips to CBD/1948 trips to CBD	.70	-------	1.10	-------	1.45	-------	1.46	-------	1.61	-------	1.64	-------	1.75	-------	1.27	-------	1.48	-------	1.46	-------
Automobile passenger trips:																				
Total intra-area trips	905	1,055	33,062	22,552	92,274	58,784	122,682	80,125	88,698	43,457	39,833	22,750	68,490	37,844	70,273	52,440	74,798	37,136	591,015	356,143
Trips to CBD	520	662	3,184	2,708	8,071	5,837	11,383	7,545	7,049	4,601	3,146	2,291	6,031	3,352	5,283	4,455	5,776	3,265	50,443	34,716
Percentage of trips to the CBD	57.5	62.7	9.6	12.0	8.7	9.9	9.3	9.4	7.9	10.6	7.9	10.1	8.8	8.9	7.5	8.5	7.7	8.8	8.5	9.7
1955 percentage/1948 percentage	.92	-------	.80	-------	.88	-------	.99	-------	.75	-------	.78	-------	.99	-------	.88	-------	.88	-------	.88	-------
1955 trips to CBD/1948 trips to CBD	.79	-------	1.18	-------	1.38	-------	1.51	-------	1.53	-------	1.37	-------	1.80	-------	1.19	-------	1.77	-------	1.45	-------
Taxi passenger trips:																				
Total intra-area trips	824	259	4,785	3,535	13,571	10,251	20,688	15,337	4,964	3,354	4,450	4,649	5,202	4,890	1,866	3,035	1,746	1,166	58,096	46,476
Trips to CBD	578	259	1,439	935	3,565	3,309	3,765	3,557	917	580	403	887	1,138	600	483	911	583	535	12,871	11,573
Percentage of trips to the CBD	70.1	100.0	30.1	26.4	26.3	32.3	18.2	23.2	18.5	17.3	9.1	19.1	21.9	12.3	25.9	30.0	33.4	45.9	22.2	24.9
1955 percentage/1948 percentage	.70	-------	1.14	-------	.81	-------	.78	-------	1.06	-------	.48	-------	1.78	-------	.87	-------	.73	-------	.89	-------
1955 trips to CBD/1948 trips to CBD	2.23	-------	1.54	-------	1.08	-------	1.06	-------	1.58	-------	.45	-------	1.90	-------	.53	-------	1.09	-------	1.11	-------
Mass-transit passenger trips:																				
Total intra-area trips	3,508	5,427	30,458	28,841	69,311	77,471	195,240	241,953	82,404	88,713	69,124	74,255	84,396	77,467	45,514	56,765	41,727	27,306	621,682	678,198
Trips to CBD	2,188	3,568	7,585	7,073	17,721	21,590	38,016	53,463	14,088	17,596	10,198	13,426	15,926	16,362	7,975	12,746	6,308	6,744	120,005	152,568
Percentage of trips to the CBD	62.4	65.7	24.9	24.5	25.6	27.9	19.5	22.1	17.1	19.8	14.8	18.1	18.9	26.1	17.5	22.5	15.1	24.7	19.3	22.5
1955 percentage/1948 percentage	.95	-------	1.02	-------	.92	-------	.88	-------	.86	-------	.82	-------	.90	-------	.78	-------	.61	-------	.86	-------
1955 trips to CBD/1948 trips to CBD	.61	-------	1.07	-------	.82	-------	.71	-------	.80	-------	.76	-------	.97	-------	.63	-------	.94	-------	.79	-------

Table 20.—Percentage distribution of trips to the central business district in 1948 and 1 55, classified according to mode of travel and sector of residence

Sectors	Automobile driver		Automobile passenger		Taxi passenger		Mass-transit passenger	
	1955	1948	1955	1948	1955	1948	1955	1948
0	0.5	1.1	1.0	1.9	4.5	2.2	1.8	2.3
1	6.6	8.8	6.3	7.8	11.2	8.1	6.3	4.6
2	16.7	16.8	16.0	16.8	27.7	28.6	14.8	14.2
3	21.6	21.5	22.6	21.7	29.3	30.7	31.7	35.1
4	13.3	12.0	14.0	13.3	7.1	5.0	11.7	11.5
5	7.4	6.6	6.2	6.6	3.1	7.7	8.5	8.8
6	12.3	10.2	12.0	9.7	8.8	5.2	13.3	10.7
7	10.9	12.5	10.5	12.8	3.8	7.9	6.6	8.4
8	10.7	10.5	11.4	9.4	4.5	4.6	5.3	4.4
Total area	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Columbia and Maryland) were considered as one group, and residents of sectors 7 and 8 (Virginia) were considered as a second group. Residents of sector zero were excluded in the analysis. In the first group, the range between the sector with the smallest and that with the largest percentage of residents CBD trips by automobile drivers was 15.9 percent in 1948 and 8.0 percent in 1955. If sector 3 is omitted from the comparison, the range is 13.5 percent in 1948 and only 2.6 percent in 1955. Group 2 indicated the same trend; there was a range of 8.0 percent in 1948 and only 1.4 percent in 1955.

The range between sectors in the percentage of residents CBD trips by mass-transit passengers decreased for group 1 from 24.6 percent in 1948 to 12.5 percent in 1955; sectors in group 2 decreased from 9.7 to 5.7 percent. The range in the percentage of trips by automobile passengers decreased in the first group of sectors, but increased slightly in the second group. The range for taxi passengers remained stable between 1948 and 1955 for both groups of sectors.

Automobile drivers

The percentage of intra-area automobile-driver trips destined to the CBD decreased for residents of each sector (table 19). The

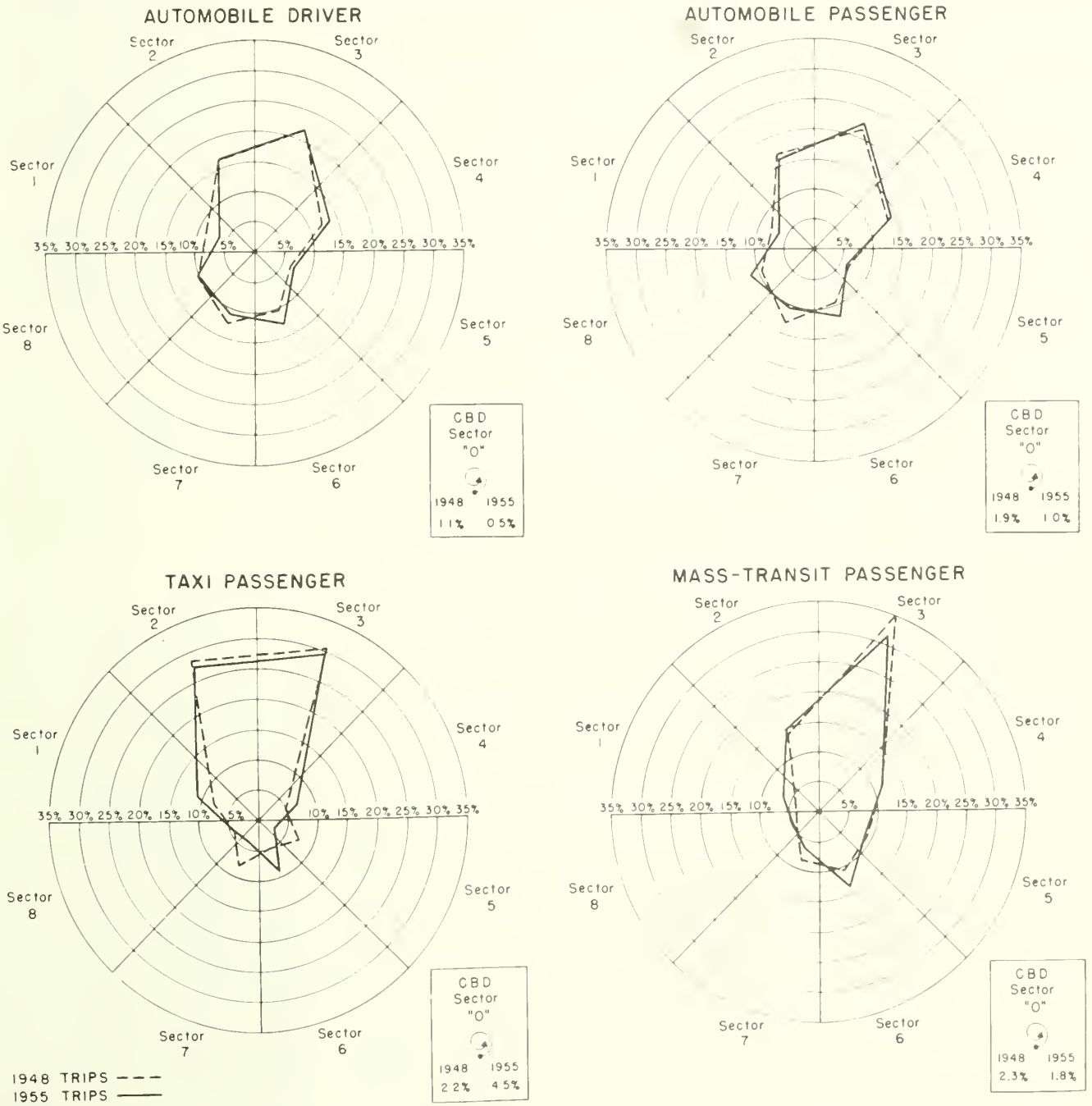

Figure 17.—Percentage distribution of automobile-driver, automobile-passenger, taxi-passenger, and mass-transit passenger trips to the CBD, according to sector of residence.

Table 21.—Number of trips per 100 dwelling units to the central business district in 1948 and 1955, classified according to mode of travel and sector of residence

Sectors	Automobile driver		Automobile passenger		Taxi passenger		Mass-transit passenger	
	1955	1948	1955	1948	1955	1948	1955	1948
0	13.3	22.7	15.6	23.9	17.4	9.4	65.8	128.9
1	24.6	37.6	14.3	20.5	6.5	7.1	34.1	53.6
2	20.4	21.2	12.0	13.1	5.3	7.4	26.3	48.3
3	13.8	12.3	8.9	7.7	2.9	3.6	29.6	54.3
4	17.1	15.8	11.1	10.7	1.4	1.4	22.1	41.0
5	14.6	13.4	7.5	8.3	1.0	3.2	24.4	48.5
6	15.2	13.1	9.0	7.6	1.7	1.4	23.9	37.0
7	17.3	17.7	10.2	11.1	.9	2.3	15.4	31.9
8	22.1	26.8	14.5	14.7	1.5	2.4	15.8	30.4
Average, total area	17.0	16.8	10.4	10.3	2.7	3.4	24.7	45.4

largest decreases occurred for the trips by residents of sectors with the greatest number of automobiles owned per dwelling unit in 1948 and 1955, sectors 1 and 8.

The pattern of distribution of automobile-driver trips by residents of each sector varied only slightly from what it had been in 1948 (table 20 and figure 17), even though automobile ownership increased by various amounts in each sector with the exception of sector zero. The percentage distribution changed within ±1.6 percentage points of what it had been in 1948 in 7 of the sectors, and in 3 sectors the change was within ±0.2 percent. Sector 3 still maintained the major peak of residents automobile-driver trips; however, because of the greater distribution of automobiles and the use thereof in comparison to taxis and mass transit, the peak in sector 3 is seen to be less extreme in both years than the distribution pattern of the latter two modes of travel.

With the increased number of automobiles owned per dwelling unit, there was a smaller difference between sectors in the number of automobile-driver trips to the CBD per dwelling unit (table 21). The range between the sector with the greatest number of trips and that with the least decreased by over 50 percent in 1955 (25.3 trips in 1948 and 10.8 in 1955).

The relationship between the number of automobiles owned per dwelling unit and the percentage of intra-area automobile-driver trips destined to the CBD changed between 1948 and 1955, as shown in figure 18. In 1948, the plotted symbols indicate no apparent trend. In 1955, however, the percentage of intra-area automobile-driver trips destined to the CBD was generally less for those sectors that had the greater number of automobiles per dwelling unit.

Although work continued to be the dominant purpose for which automobile-driver trips were made to the CBD, the change in the percentage distribution of trip purposes for each of the sectors indicated a general tendency for work trips to decrease in relative importance while "other" trips increased (table 16). Sectors 1 and 7, the two sectors with the highest percentage of 1948 CBD automobile-driver trips made for "other" purposes, were the exceptions.

Another change which occurred was the decrease in the range of purpose distribution for work and "other" purposes between five of the sectors (table 16). In 1948 residents of sectors 1, 2, 4, 7, and 8 had a range in the percentage of their CBD automobile-driver trips for work of 13.4 percent; for "other" purposes, it was 11.7 percent. In 1955 these ranges had decreased to 3.6 percent for work and 1.0 percent for "other" trips. It is interesting to note that these sectors also included the residents who had the highest average incomes in the study area as well as the greatest number of automobiles owned per dwelling unit.

Automobile passengers

The relative changes between 1948 and 1955 in the distribution pattern of sector residents CBD automobile-passenger trips were generally small (table 20 and figure 17); six of the

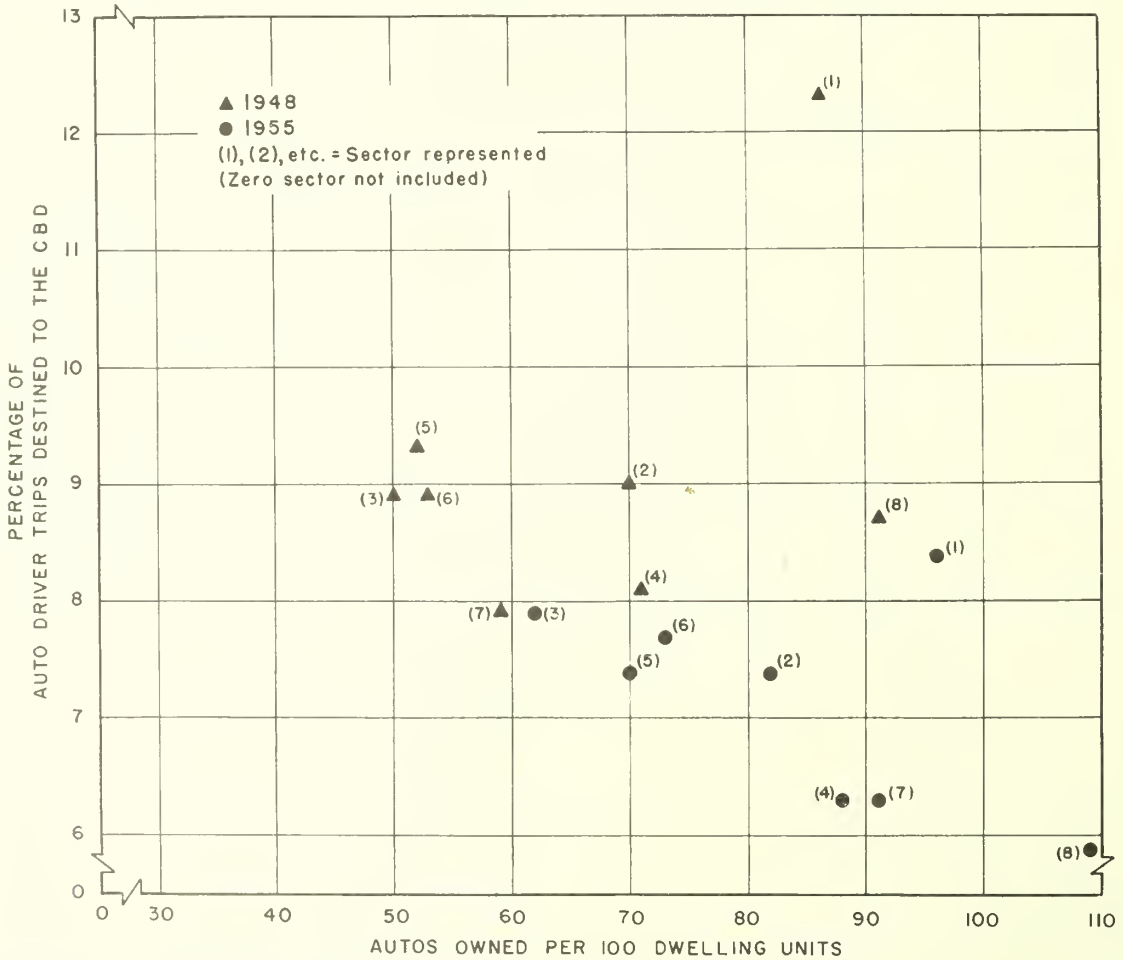

Figure 18.—Percentage of CBD automobile-driver trips to total area trips related to the number of automobiles owned per 100 dwelling units, according to sector of residence.

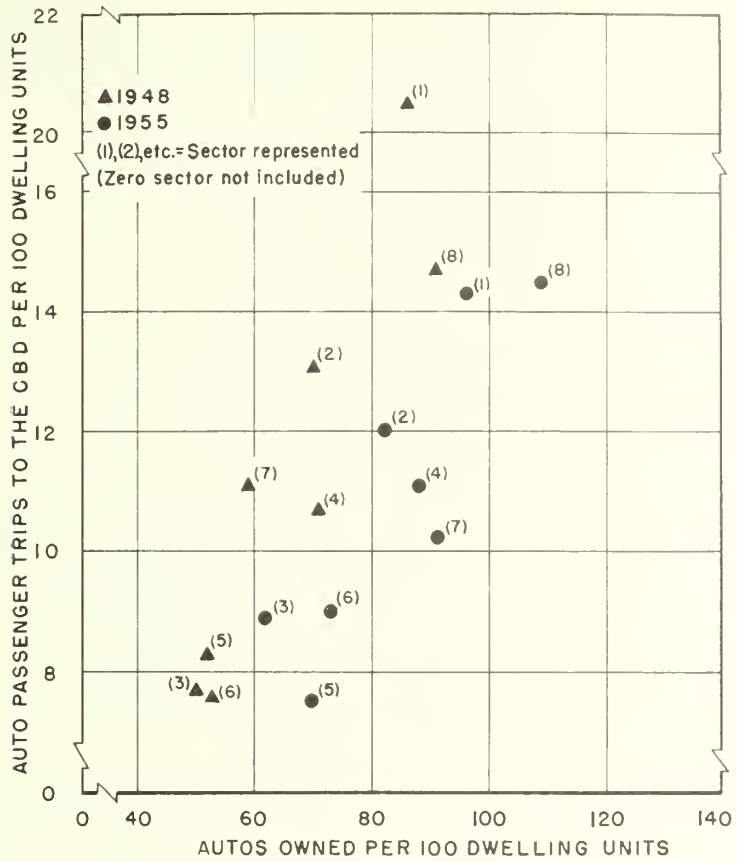

Figure 19.—Relation of the number of CBD automobile-passenger trips per 100 dwelling units to the number of automobiles owned per 100 dwelling units, according to sector of residence.

sectors had a change that was within ±1.5 percentage points of the 1948 figure. As indicated in figure 17, the distribution of residents trips in both years showed a pattern very similar to that of automobile-driver trips; and although sector 3 contained the major peak of automobile-passenger trip distribution, as in automobile-driver trips, the peak was not so extreme as the distribution of mass-transit passenger trips.

The relationship between the number of automobiles owned and the number of automobile-passenger trips to the CBD per dwelling unit remained approximately the same, even though the number of automobiles owned increased in each sector except sector zero, and the range of trips per dwelling unit had decreased (fig. 19). The trend in 1955 as in 1948 was for the greater number of automobile-passenger trips to the CBD per dwelling unit to be found in those sectors having the largest number of automobiles per dwelling unit.

The proportion of intra-area automobile-passenger trips to the CBD (table 19) decreased in each of the sectors. Residents of sectors 3 and 6, however, maintained approximately the same percentage. They also had the smallest amount of change in the percentage of their intra-area automobile-driver trips destined to the CBD.

While residents of most sectors made an increasing proportion of their CBD automobile-driver trips for "other" purposes and a decreasing percentage for work, residents automobile-passenger trips showed an opposite tendency (table 16). As mentioned earlier, car pools were much more common in 1955 than in 1948. In 1948, residents of only one sector made over 60 percent of their CBD automobile-passenger trips to work. By 1955 residents of 7 sectors were in the over 60 percent group, and 4 of these sectors exceeded 70 percent.

Taxi passengers

There appears to have been a slightly greater shifting between sectors in the percentage distribution of residents CBD taxi-passenger trips than was indicated in the distribution of residents trips by the other modes of travel; 4 of the 9 sectors had a percentage change greater than ±3.0 percentage points (table 20 and figure 17). Nevertheless, in 1955 as in 1948, approximately 60 percent of the taxi-passenger trips destined to the CBD were made by residents of adjacent sectors 2 and 3. As taxi-passenger trips made by visitors were not reported in this study, sector zero does not appear to have a large percentage of taxi-passenger trips.

Except for sectors 0, 4, and 6, taxi-passenger trips per dwelling unit decreased between 1948 and 1955 (table 21). With this decrease, although the difference between sectors was small, the geographical relationship between residents of sectors 1 through 5 appeared to be greater than was apparent in 1948. In 1955 the number of taxi-passenger trips per dwelling unit decreased continuously in the sectors from 1 through 5. This transition again appeared to be one from sectors with high average family incomes to one of lower average family incomes. In addition, the relatively lower number of taxi-passenger trips per dwelling unit made by Virginia residents may partially be due to the greater cost of taxi transportation to the CBD than for similar distances in the District of Columbia.

The percentage of intra-area taxi-passenger trips destined to the CBD decreased for residents of each sector, with the exception of sectors 1, 4, and 6. It is not known why the latter sectors did not have the same general trend as the others, for an increase in the proportion of residents trips by any of the modes was unusual. The only other increase was mass-transit passenger trips destined to the CBD by residents of sector 1.

Although the percentage distribution of trip purposes for which CBD taxi-passenger trips were made shifted in each of the sectors, the most significant change was the decrease in the percentage of "other" purpose residents trips in 8 of the 9 sectors (table 16). Taxi-passenger work trips increased in 5 of the 9 sectors, and in 6 of the 9 sectors, 56 percent or more of the taxi-passenger trips were to work.

Mass-transit passengers

As in the other modes of travel, there was a shifting between sectors in the relative distribution of total residents mass-transit passenger trips to the CBD (table 20 and figure 17). Nevertheless, the pattern of distribution changed only slightly as 7 of the sectors were within ±2.0 percentage points of the 1948 figure and 4 of the sectors were within ±0.5 percent. The extremely high volume of trips made by residents of sector 3 can be attributed to the concentration of population in apartments, row houses, and boarding houses, and the relatively adequate transit service for this sector.

Unlike other modes of travel to the CBD, the number of mass-transit passenger trips per dwelling unit decreased in each sector between 1948 and 1955 (table 21). As a result, there was a growing similarity in the number of trips per dwelling unit in all sectors. The area in which the most important decrease took place occurred between sectors 2 through 6. In 1948, the range between the sector with the greatest and that with the fewest number of mass-transit passenger trips to the CBD per dwelling unit was 17.3 trips. By 1955 the range was only 7.5. If sector 3 is excluded in the comparison, the range is 11.5 trips in 1948 and 4.2 in 1955.

Of the percentage of intra-area mass-transit trips destined to the CBD, it was found that only residents of sector 1 maintained approximately the same percentage in 1955 as in 1948 (table 19). The relative decreases that did take place in the other sectors, however, appeared to have a geographical relationship with the CBD. A comparison beginning with sector 2 and moving around the CBD through sector 8, with the exception of sector 6, would show that a progressive increase in the relative changes in orientation took place between 1948 and 1955 (table 19).

Trips to work remained the principal purpose for which sector residents made mass-transit passenger trips to the CBD (table 16). By 1955 residents of each sector, excluding sector zero, were making 60 percent or more of their CBD mass-transit passenger trips to work. Trips made by residents of sectors 7 and 8 are especially noteworthy. By 1955 these residents of Virginia were making approximately 4 out of 5 mass-transit passenger trips for the purpose of work, which indicated the very limited use of mass transit to the CBD for all the other purposes.

New Publications

The *Annual Report of the Bureau of Public Roads, Fiscal Year 1958*, is now available from the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 30 cents a copy.

During the fiscal year, the greatly expanded, long-range highway construction program launched by the Federal-Aid Highway Act of 1956 reached full stride. Completions of all classes of Federal-aid and Federal projects provided improvements on 24,204 miles of roads and streets. Projects for the construction of 35,698 miles of improvements were programed and contracts were awarded for 25,912 miles of road and street improvements.

Federal-aid operations of the year were supported largely with funds authorized by the Federal-Aid Highway Act of 1956, although minor balances from previous authorizations were also being used. On August 1, 1957, Federal-aid funds for the fiscal year 1959, authorized by the 1956 act and amounting to $2.875 billion, were apportioned to the States. An additional $600 million authorized by the Federal-Aid Highway Act of 1958 was appertioned on April 16, 1958. The total of Federal aid apportioned to the States since passage of the 1956 act was thus brought to $7.150 billion.

A notable event of the year was the passage of the Federal-Aid Highway Act of 1958. Following traditional practice, it authorized Federal-aid primary, secondary, and urban funds, and funds for constructon in Federal lands, for 2 fiscal years, 1960 and 1961. The act increased the Interstate authorizations made in the 1956 act for the 3 fiscal years 1959–61, and set aside the "pay-as-you-go" clause in the 1956 act so as to permit apportionment of the full amounts authorized for fiscal years 1959 and 1960. The act also provided for control of advertising along the Interstate System.

Recognizing the value of highway construction in the Nation's antirecession efforts, the Congress authorized under the 1958 act $400 million for immediate apportionment to the States for primary, secondary, and urban work. These funds, to be matched on a two-thirds Federal, one-third State basis rather than the usual 50–50 ratio, were required to be placed under contract by December 1, 1958, with work scheduled for completion by December 1, 1959.

The Federal-aid primary, secondary, and urban highway improvement programs progressed on a larger scale than ever before, but the attention of the public was attracted to the Interstate System program. Thousands of people were directly concerned, as hearings on choices of location were held and property for right-of-way was bought. The State highway departments cooperated closely with local officials and planning bodies in selecting locations in the best overall public interest.

During the fiscal year 696 miles of pavement were completed on the Interstate System, but far more was accomplished in surveys and plans, right-of-way acquisition, and grading and drainage construction. At the end of the year $3.6 billion worth of work was underway or scheduled to start soon.

Two major reports relating to the Interstate System were presented to the Congress during the year. One of these, an estimate of the cost of completing the system, involved a detailed survey by the States and Public Roads of the 38,548 miles of routes then included in the system. It was found that Federal and State matching financing required after July 1, 1956 amounted to $37.6 billion, as compared with the $27.6 billion available from authorizations of the Federal-Aid Highway Act of 1954 and 1956 together with State matching funds.

The other report was prepared in connection with Congress' declaration of intent to determine whether or not the Federal Government should reimburse the States for toll and free highways on the Interstate System built between 1947 and 1957. It was found that 10,859 miles of the system met the criteria for consideration for reimbursement. Construction of these facilities, which included 1,950 miles of toll roads, had cost $6.09 billion.

Also available from the Superintendent of Documents, at 75 cents a copy, is the revised edition of the publication entitled *Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline.*

This publication was prepared by the Photogrammetry for Highways Committee with active participation by the Bureau of Public Roads. The Photogrammetry for Highways Committee is jointly sponsored by the American Society of Photogrammetry and The American Congress on Surveying and Mapping.

The purpose of the publication is to aid highway officials in establishing specifications for the procurement by contract of photogrammetric and aerial survey services. Under the provisions of the Federal-Aid Highway Act of 1956, the Secretary of Commerce may authorize the use of photogrammetric methods in mapping, and the utilization of commercial enterprise for such services.

The specifications are a reference guide outline, and should be modified to fulfill special requirements not discussed in detail in this publication.

Lost Mixing Time of Dual Drum Pavers: A Motion Picture

The Bureau of Public Roads, U.S. Department of Commerce, recently produced a new motion picture, *Lost mixing time of dual drum pavers.* The film, based on extensive studies of portland cement concrete paving, conducted by Public Roads, highlights the importance of the simultaneous mixing interval in dual drum pavers in meeting mixing time specifications. It shows some trouble spots and the significance of proper adjustments to the batchmeter. The film had its premier showing at the annual meeting of the Highway Research Board of the National Academy of Sciences on January 6, 1959.

Contractors and engineers will have a particular interest in the contrasting scenes which illustrate why two successive batches of portland cement concrete from a dual drum paver are frequently mixed for a different length of time. The film also illustrates how minimum mixing time for which the batchmeter is set can be measured. Many of the illustrations are animated cutaway views of concrete being mixed within the mixing drums which, followed by live action scenes of field construction operations, make it possible to follow the mixing process as it actually occurs on the job.

In the tremendous highway construction program now under way, the Bureau of Public Roads has a vital interest in improvement of job management which results in the highest rate of production consistent with quality. Greater attention to what may superficially seem to be trivial problems permits the contractor to offer lower bids on construction work without cutting his profits, and provides the engineer with greater assurance that the end product will meet his standards of quality.

Lost mixing time of dual drum pavers is a 16-mm. sound and color film with a running time of 30 minutes. Prints may be borrowed for showings by any responsible organization by request addressed to Mr. Ray B. Dame, Chief, Photographic Services, Bureau of Public Roads, Washington 25, D.C. There is no charge except for the express or postage fees. Requests should be sent well in advance of the desired showing, and alternate dates should be given if possible. Prompt return after each showing is necessary, so that all requested bookings may be fulfilled.

Prints of *Lost mixing time of dual drum pavers* may be purchased at $127.45 per copy, the price including film, reel, can, and shipping container, and postage within the United States. Inquiries should be addressed to Mr. Ray B. Dame, Chief, Photographic Services, Bureau of Public Roads, Washington 25, D.C. Payment should not be sent with the inquiry.

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Annual Reports of the Bureau of Public Roads:
1950, 25 cents. 1953 (out of print). 1956, 25 cents.
1951, 35 cents. 1954 (out of print). 1957 (out of print).
1952, 25 cents. 1955, 25 cents. 1958, 30 cents.

PUBLICATIONS

A Report of Factors for Use in Apportioning Funds for the National System of Interstate and Defense Highways, House Document No. 300 (1958). 15 cents.

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). Out of print.

Consideration for Reimbursement for Certain Highways on the Interstate System, House Document No. 301 (1958). 15 cents.

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First Progress Report of the Highway Cost Allocation Study, House Document No. 106 (1957). 35 cents.

General Location of the National System of Interstate Highways, Including All Additional Routes at Urban Areas Designated in September 1955. 55 cents.

Highway Bond Calculations (1936). 10 cents.

Highway Capacity Manual (1950). $1.00.

Highway Needs of the National Defense, House Document No. 249 (1949). 50 cents.

Highway Practice in the United States of America (1949). Out of print.

Highway Statistics (annual):
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Mathematical Theory of Vibration in Suspension Bridges (1950). $1.25.

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Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways, 1958: a reference guide outline. 75 cents.

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Standard Plans for Highway Bridge Superstructures (1956). $1.75.

Taxation of Motor Vehicles in 1932. 35 cents.

Tire Wear and Tire Failures on Various Road Surfaces (1943). 10 cents.

Transition Curves for Highways (1940). $1.75.

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Bibliography on Automobile Parking in the United States (1946).

Bibliography on Highway Lighting (1937).

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Express Highways in the United States: a Bibliography (1945).

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OL. 30, NO. 8 JUNE 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

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Newly constructed riveted plate girder bridge over the Willamette River, 3 miles south of Newberg, Oreg., on State Route 219. This structure, measuring 1,090 feet and costing $1,085,000, is located on the Federal-aid secondary system.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 8 *June 1959*

C. M. Billingsley, *Editor*

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Washington 25, D.C.

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IN THIS ISSUE

Correlation Between Chemical and Mortar Bar Tests for Potential Alkali Reactivity of Concrete Aggregates 177

Home-Interview Traffic Surveys and Related Research Activities 185

Pretreatment of Soils and Clays for Measuring External Surface Area by Glycerol Retention . 187

Nonelastic Deformations in Continuous Concrete Structures 191

New Publications 196

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

U.S. DEPARTMENT OF COMMERCE
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Correlation Between Chemical and Mortar Bar Tests for Potential Alkali Reactivity of Concrete Aggregates

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

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Certain siliceous constituents of aggregates can react with cement alkalies to cause abnormal expansion and cracking in concrete. The American Society for Testing Materials (ASTM) mortar bar expansion test is generally considered to be the most reliable method available for determining the potential alkali reactivity of such aggregates. Since this test requires considerable time, a quick chemical test was also adopted by the ASTM in order to obtain a rapid evaluation of concrete aggregates. This investigation was made to determine the degree of correlation between the two test methods.

Fifty-two concrete aggregates were evaluated by means of the rapid chemical test, and the results generally showed good correlation with the data obtained in the mortar bar tests. However, the chemical test does not always give reliable results for some aggregates because of the interference of certain minerals. An empirical division of the chemical test results is proposed which would serve to isolate those aggregates which may contain such interferences, and thereby indicate the need for further tests.

Some aggregates showed a high degree of reactivity in the chemical test but did not produce excessive mortar bar expansion. An additional empirical division of the chemical test results is suggested which would separate chemically reactive aggregates into two groups. Such a separation would eliminate the need of supplementary mortar bar tests on many reactive aggregates. Chemical test results which fall into one such group are indicative of very highly reactive but not necessarily expansion-producing aggregates. Special mortar bar tests are necessary for such materials.

The general chemical test criteria established by this study for separating reactive from nonreactive aggregates agree fairly well with those proposed by the American Society for Testing Materials and the U.S. Corps of Engineers. The present ASTM chemical test method permits two titration procedures for the determination of alkalinity reduction. Neither procedure showed any distinct advantage over the other as a means for evaluating the potential reactivity of aggregates.

BEFORE 1940, most aggregates were usually considered to be chemically inert when used in concrete. However, since that time, it has been clearly demonstrated that certain siliceous constituents, present in some aggregates, can react with the alkalies in cements to cause abnormal expansion and cracking in concrete.

The most reliable laboratory test now available for determining the potential alkali reactivity of concrete aggregates is considered to be the mortar bar expansion test (*1*).[2] This test has been adopted as tentative standard method C227 by the American Society for Testing Materials (*2*). Because of the time required to obtain reliable information by this test method, extensive investigations have been made to develop a more rapid test. A rapid chemical test was developed by Mielenz and colleagues (*3*) and was later adopted by the ASTM as tentative standard method C289 (*2*). Briefly, the chemical test measures the amount of reaction during 24 hours at 80° C. between a 1-normal sodium hydroxide solution and aggregate that has been crushed and sieved to definite size. At the end of the reaction period, the mixture is filtered and the solution is analyzed to determine its final alkalinity and silica content, the latter arising as a result of the partial dissolution of the aggregate. These values are then used to determine the potential reactivity of the aggregate with alkalies in portland cement concrete.

The reliability of the chemical test was established by the original authors of the method, mainly on the basis of an empirical correlation with mortar bar expansion test results. Unfortunately, many investigators have subsequently reported discrepancies in evaluating concrete aggregates by both test methods.

A search of the literature failed to reveal any additional published work in which a large number of aggregates were evaluated by both test methods. Comparisons of a few aggregates have been reported by several investigators (*4, 5, 6*). Two other reports (*7, 9*) showed considerable comparative data, but most of the results were either reported earlier by Mielenz and colleagues (*3*), or were based upon modifications of the presently used chemical test.

Consequently, the Bureau of Public Roads initiated a study of a large number of aggregates in order to compare the results of both test methods. A specific objective of the study was to determine the degree of correlation between the results of the quick chemical test and the mortar bar test. It was anticipated that the present criteria of the chemical test could be suitably altered or extended in order to make better use of the test results, thereby eliminating the need for further extensive testing of many aggregates.

Conclusions

1. The chemical test for the potential reactivity of aggregates, ASTM C289–57T, generally shows good correlation with the results obtained by mortar bar tests.

2. The dividing line, established by this study for separating reactive from nonreactive aggregates by means of the chemical test,

[1] The authors wish to acknowledge the helpful assistance of Messrs. Donald O. Woolf, Russel H. Brink, and George Markmann. Messrs. Woolf and Brink gave technical advice and supplied physical test data; Mr. Markmann assisted in the preparation of test samples and in the assembling of the mortar bar test data. This article was presented at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959.

[2] Italic numbers in parentheses refer to the list of references on page 184.

Table 1.—Results of mortar bar expansion and chemical tests on concrete aggregates

Sample No.	Source of aggregate	Type of aggregate	Nature of aggregate [1]	Mortar bar test data [2]						Chemical test data [4]			Remarks
				Cement used				Expansion at 1 year	Evaluation of potential reactivity [3]	Alkalinity reduction (Rc)		Dissolved silica (Sc)	
				Brand and lot No.	Na_2O	K_2O	Equivalent Na_2O			Single-end point	Double-end point		
					Percent	*Percent*	*Percent*	*Percent*		*mM/L*	*mM/L*	*mM/L*	
1	Burnt River, Oreg	Sand	R, Sc, G, Qz, Fe	A-1	0.11	0.97	0.75	0.08	Q	134	151	122	70° F. storage.
1	do	Gravel	R, B, Sy, Sc, less than 1% Qz	C-1	.27	.70	.73	.04	NR				
2	Klamath Falls, Klamath County, Oreg	Rock	B	B	.20	.47	.51	.12	R	137	159	160	Medium alkali cement in bars.
3	Near Pocatello, Power County, Idaho	Gravel	Qz, Felds-Qz, Calc-S, Q, B, R.	A-2	.14	1.04	.82	.11	R	45.3	63.7	106	
4	Idaho Falls, Bonneville County, Idaho	do	Qz, S, Felds-Qz, Calc-S, L, G	A-2	.14	1.04	.82	.12	R	49.2	70.8	108	Sand fraction was reactive with same cement.
4	do	do	Qz, S, Felds-Qz, Calc-S, L, G	E	.21	.60	.60	.04	NR				
5	Pocatello, Bannock County, Idaho	Sand	Qz, Felds-Qz, L; some Q and S	A-2	.14	1.04	.82	[5].05	Q	86.3	111	136	Bars reactive with 1% NaOH.
6	McCammon, Bannock County, Idaho	do	Q, Qz, L; some S and Ma	A-2	.14	1.04	.82	.04	NR	126	152	273	Do.
7	Teton, Fremont County, Idaho	do	Q, G, Fe, Qz, O, Calc fragments	A-2	.14	1.04	.82	.07	NR	116	129	466	Bars reactive with 1% NaOH, and at 2 years without NaOH.
7	do	Gravel	G, Qz, Felds-Qz; some Q, R, and L.	A-2	.14	1.04	.82	.06	NR				
8	Blackfoot, Bingham County, Idaho	Sand	O, Q, Qz; some S and Ma	A-2	.14	1.04	.82	.35	R	108	126	354	
8	do	Gravel	Qz, Felds-Q, S	A-2	.14	1.04	.82	.12	R				
9	Menan, Jefferson County, Idaho	Sand	Q, O, Fe; some S and Ma	A-2	.14	1.04	.82	.15	R	67.1	82.8	217	
9	do	Gravel	Felds-Qz, S, Fe; some Q, G, Gr.	A-2	.14	1.04	.82	.34	R				
10	Rupert, Minidoka County, Idaho	Sand	Qz, S, Fe, Q; some O, Pum, Ma.	A-2	.14	1.04	.82	.08	Q	148	156	302	Bars reactive with 1% NaOH.
10	do	Gravel	R, L; some S, Qz, and Q	A-2	.14	1.04	.82	.17	R				
11	Rexburg, Madison County, Idaho	Sand	Q, O, Fc; some Qz	A-2	.14	1.04	.82	.09	Q	130	151	476	Do.
11	do	Gravel	R, G; some B, O, Felds-S, Q	A-2	.14	1.04	.82	.07	NR				
12	Salt Lake City, Salt Lake County, Utah	do	Qz, Calc-S, L; some Q, Gn, Dol and G.	C-2	.18	.87	.75	.11	R	66.8	110	26.7	Dolomite present. Bars reactive with medium alkali cement.
13	Hoover's Siding, Chesterfield County, S.C.	Sand	Q; some S	A-2	.14	1.04	.82	.05	NR	9.2	17.6	20.7	Bars reactive in 2 years.
14	Near Sapinero, Gunnison County, Colo.	Gravel	B, R, volcanics with G, dark Aph.	C-2	.18	.87	.75	.32	R	215	236	472	
15	Sta. 621, Loveland Fremont Pass, Colo.	Sand	Q, G, and dark Aph	C-2	.18	.87	.75	.04	NR	40.4	54.9	20.7	
16	Near Rand, Jackson County, Colo	do	Q, G, M	C-2	.18	.87	.75	.03	NR	62.4	77.9	27	
17	Snake River, near Hoback, Teton County, Wyo.	Gravel	L, Qz, S	C-2	.18	.87	.75	.06	NR	45.2	57.2	66	Sand fraction showed questionable reactivity in 2 years.
18	Oral, Fall River County, S. Dak	do	L, S, Calc-S, Qz	C-2	.18	.87	.75	.03	NR	163	233	74.	Limestone present.
19	Hill City, Pennington County, S. Dak	Rock	Micaceous Qz	C-2	.18	.87	.75	.04	NR	36.6	55.2	29.8	
20	Snake River, Teton County, Wyo	Gravel	Qz, Calc-S, Fe, R, B	C-2	.18	.87	.75	.02	NR	178	199	537	Bars reactive with 1% NaOH.
21	Dayton, Sheridan County, Wyo	Rock	Altered G	C-2	.18	.87	.75	.03	NR	31.1	44.4	16.8	
22	do	Gravel	Qz, L, G	C-2	.18	.87	.75	.03	NR	135	209	84	Limestone present.
23	do	Sand	G, Q, L	C-2	.18	.87	.75	.04	NR	73.1	111	18.8	Do.
24	Nederland, Boulder County, Colo	Gravel	G, Gn	C-2	.18	.87	.75	.03	NR	35.7	42.9	39.4	
25	do	Sand	G; some Q and Gn	C-2	.18	.87	.75	.05	NR	43.7	60.9	22.4	
26	St. Vrains, Weld County, Colo	Gravel	G, Gn	C-2	.18	.87	.75	.04	NR	20.4	31.9	22	
27	Cameo, Delta County, Colo	Sand	Q, Fc, dark Aph; some F	C-2	.18	.87	.75	.04	NR	135	182	38.6	
28	Near Mesa, Mesa County, Colo	Rock	Olivine B	C-2	.18	.87	.75	.07	NR	89.5	110	128	Questionable reactivity in 2 years.
29	Near Reserve, Catron County, N. Mex.	Gravel	L, B, An; some G	C-2	.18	.87	.75	.10	R	176	199	236	Limestone present.
30	El Paso, El Paso County, Tex	Sand	B, Q; some F	C-2	.18	.87	.75	.62	R	79.8	91	234	
31	Dry Canyon, near LaLuz, Otero County, N. Mex.	Rock	L	C-2	.18	.87	.75	.08	Q	45.5	68	11.5	Limestone present; dolomite and magnesite absent.
32	Ruidosa River, near Hondo, Lincoln County, N. Mex.	Gravel	L, S, Qz, Fe	C-2	.18	.87	.75	.02	NR	106	179	13.6	Limestone present.
33	do	Sand	Dark Aph, G, L; some Di	C-2	.18	.87	.75	.02	NR	127	172	16.4	Do.
34	Cheraw, Chesterfield County, S.C	Gravel		C-2	.18	.87	.75	.10	R	8.3	11.5	24.3	Bars nonreactive at 70° F. storage or with 1% NaOH admixture.
35	Columbia, Richland County, S.C	Rock	G	C-2	.18	.87	.75	.08	Q	7	14.0	15.6	Do.
36	Beverly, Pickens County, S.C	do	Biotite G	C-2	.18	.87	.75	.06	NR	10.6	23.5	14.8	
37	Blairs, Fairfield County, S.C	do	G	C-2	.18	.87	.75	.06	NR	9.3	17.6	13.1	
38	Chattahoochie, Gadsden County, Fla	Sand		D-1	.32	.95	.94	.04	NR	16.1	25.1	19.1	100° F. storage.
39	Columbus, Lowndes County, Miss	Gravel	C	D-2	.33	.86	.90	.03	NR	154	178	454	100° F. storage. Concrete bars.
40	Dixieland, Russell County, Ala	Sand		D-2	.33	.86	.90	.04	NR	13.7	20.4	20.1	100° F. storage.
41	Montgomery, Montgomery County, Ala	do		D-2	.33	.86	.90	.05	NR	17.2	24	27.9	Do.
42	Chattanooga, Hamilton County, Tenn	do		D-2	.33	.86	.90	.08	Q	39.3	56.1	62	Do.
43	Camak, Warren County, Ga	Rock		D-2	.33	.86	.90	.03	NR	9.6	18.6	17.2	Do.
44	Near Pocatello, Power County, Idaho	Sand	Q, Qz; some S and Sc	A-2	.14	1.04	.82	.12	R	66.3	85.8	169	
44	do	Gravel	Qz, Felds-Qz; some S and Q	A-2	.14	1.04	.82	.14	R				
45	San Ramon Quarry, Nicaragua	Rock		D-1	.32	.95	.94	.22	R	78.7	106	124	Do.
46	Clinton Point, Dutchess County, N.Y.	do	Dol	D-2	.33	.86	.90	.05	NR	98.5	173	10.6	100° F. storage. Dolomite.
47	San Marcos Bridge (right side), Salvador	Sand		D-2	.33	.86	.90	.03	NR	140	166	131	100° F. storage.
48	San Marcos Bridge (left side), Salvador	Rock		D-2	.33	.86	.90	.14	R	130	149	212	Do.
49	San Marcos Bridge (right side), Salvador	do		D-2	.33	.86	.90	.09	Q	137	151	340	Do.
50	Pioneer Pit, Crackerville Junc., Mont.	Gravel	Qz, fine-grained volcanics, G	D-3	.20	.80	.73	[6].02	NR	124	140	226	100° F. storage. Concrete bars.
51	Tierney Bros., Crackerville Junc., Mont.	do	Qz, Ser-volcanics, G, S, and R	D-3	.20	.80	.73	[6].02	NR	198	216	373	Do.
52	Ottawa, Ill	Sand	Q	C-2	.18	.87	.75	.04	NR	.7	4.1	14.9	Bars of Ottawa sand (30–100).

[1] Abbreviations used: An, andesite; Aph, aphanites; B, basalt; C, chert; Calc, calcareous; D, diabase; Di, diorite; Dol, dolomite; F, feldspar; Fe, felsite; Felds, feldspathic; G, granite; Gn, Gneiss; Gr, Greenstone; L, limestone; M, mica; Ma, magnetite; O, obsidian; Pum, pumice; Q, quartz; Qz, quartzite; R, rhyolite; S, sandstone; Sc, schist; Ser, sericitic; and Sy, syenite.

[2] Sand and coarse aggregate sieved and crushed, respectively, to pass No. 4 sieve. Mortar bars, measuring 1 by 1 by 11 inches and prepared with a 1 to 2 mix, were stored in moist air at 120° F. Exceptions to these conditions are noted in the last column of the table.

[3] Criteria were based on mortar bar expansion data at 1 year. "Q" (questionable) represents results in doubt or 1 year expansions of 0.08 to 0.09 percent; "R" (reactive) represents expansions of 0.10 percent or more, and "NR" (nonreactive) represents expansions of 0.07 percent or less.

[4] ASTM Method C 289–57T.

[5] Expansion data at 6 months. Specimens broke after this valuo was determined.

[6] Data at 2 years.

agrees fairly well with those proposed by the ASTM in Specification C 33 and the U.S. Corps of Engineers.

3. Neither the single- nor double-end point titration, permitted by the ASTM chemical test method, offers any distinct advantage in evaluating the potential reactivity of aggregates.

4. Reliable results are not always obtained in the chemical test when certain carbonates or serpentine minerals are present. An empirical division of the chemical test results is proposed which would serve to isolate such interferences, and thereby indicate which aggregates require further study. This procedure would eliminate the need for additional time-consuming tests on many satisfactory aggregates.

5. Highly reactive aggregates may be classified as reactive in the chemical test and yet fail to produce mortar expansion. An additional empirical division of the chemical test results is suggested which would serve to identify such aggregates. It is believed that many such aggregates may actually become dangerous in concrete if diluted with inert aggregates. The proposed division would focus attention upon those aggregates requiring special investigation.

6. A modification of the ASTM main boundary line is proposed as part of the general criteria for chemical tests. The adjusted line conforms more realistically to available data.

Procedure

Chemical tests

Fifty-two concrete aggregate samples were prepared and tested in accordance with ASTM method C289(*2*), except that the sodium hydroxide solution was prepared by standardizing with standard hydrochloric acid. The acid was standardized against reagent grade sodium carbonate using methyl purple indicator.

The reaction containers used were fabricated from 18–8 stainless steel and fitted with unvulcanized neoprene gaskets. Ethylene glycol was used for the constant temperature liquid bath. Filtration vacuum was maintained at 15 inches of mercury by means of a Cartesian Manostat. Immediately following filtration, the diluted solutions were stored in polyethylene containers and sealed until analyzed.

The reduction in alkalinity was determined by means of both the single- and double-end points given in the ASTM method. The single-end point titration involves a neutralization with standard acid, using phenolphthalein as the indicator, and measures the alkalinity produced by all the hydroxide ions as well as part of the carbonate ions present. In the double-end point procedure, the titration of the same sample is continued to the methyl orange-end point which on calculation gives a measure of the hydroxide ions only.

Dissolved silica was determined colorimetrically with a Beckman DU spectrophotometer using 1 cm. cells. The readings were made at exactly 5 minutes after adding the oxalic acid reagent.

Mortar bar tests

Mortar bar test data had been obtained for these same aggregates over a considerable number of years and, in a large number of cases, well before the development of the ASTM standard method. Consequently, the procedures used in the preparation and storage of the test bars varied somewhat. The details and conditions of test are shown in table 1.

Results

The results of the mortar bar and chemical tests on representative samples of aggregate are tabulated in table 1. Included in the table are the sources and petrographic analysis for most of the aggregates. Table 1 also shows an evaluation of the potential reactivity of the aggregates, based upon the amount of expansion exhibited by mortar bars at an age of 1 year. Expansions of 0.10 percent or more were considered to be indicative of reactive aggregates as suggested by ASTM Specification C33–57T, section 4c(*2*). Expansions of less than 0.08 percent were considered to be indicative of nonreactive materials, while aggregates having expansions of 0.08 or 0.09 percent were classified as questionable. The latter criteria conform to the established practice of the Bureau of Public Roads.

Some of the aggregates studied consisted of sand and gravel fractions which were evaluated separately in the mortar bar test. However, because of limited amounts of material available after the mortar bar tests, several such companion fractions were combined and prepared as a single sample for the chemical test. In such cases, no special problem arose where the mortar bar data for companion fractions indicated the same degree of reactivity or nonreactivity. In a few cases, however, companion fractions were classified into different categories of mortar bar reactivity. In these few cases, sample Nos. 1, 4, 10, and 11, the result representing the greatest degree of mortar bar reactivity was arbitrarily selected for comparison with the chemical test result.

Table 2 shows only the chemical test results on several additional materials. These were special materials which included carbonate minerals and well-known reactive materials.

The results of the chemical test for each material, shown in tables 1 and 2, are presented graphically in figures 1 and 2. The units used in both graphs and the method of presentation are in accordance with ASTM method C289(*2*), and as suggested by the original authors of the chemical test (*3*). Figure 1 represents the results obtained by using the single-end point in the alkalinity reduction titration, while figure 2 shows the results obtained by means of the double-end point titration.

In figures 1 and 2, each chemical test result is represented by a symbol to denote both the cement used and the degree of reactivity found by the mortar bar test.

Discussion of Test Results

In figures 1 and 2, a solid line was drawn to separate the bulk of the reactive from the nonreactive aggregates. The classification for degree of reactivity was based upon the mortar bar test results as previously described. This manner of separation was proposed by Mielenz and colleagues (*3*) and is included in a recent revision of ASTM method C–289. Once a line of demarcation is established, it serves as a criterion for future chemical tests in evaluating additional aggregates. Chemical test results to the right of the solid line should be indicative of potentially reactive aggregates, while results to the left of the solid line should represent nonreactive aggregates.

Line of separation

Figures 1 and 2 show two lines of separation. The solid line in each case applies to the data obtained in this study, whereas the dashed line represents other published criteria.

In figure 1, the dashed line is based upon the calculations suggested in ASTM Specification C33–57T(*2*). Both the dashed and solid lines shown in this graph coincide fairly well; thus the criteria established by this study are in substantial agreement with those suggested by the ASTM specification. A slight spread exists between the two lines in the lower part of the graph. The position of the solid line in this area was dictated by the plotted results of sample Nos. 17, 42, and 34. Sample No. 34 showed a mortar bar expansion of 0.10 percent in 1 year, the minimum value necessary for classifying an aggregate as re-

Table 2.—Results of chemical reactivity test on special materials

Sample identification	Nature of aggregate	Chemical test data [1]		
		Alkalinity reduction (Rc)		Dissolved silica (Sc)
		Single-end point	Double-end point	
		mM/L	*mM/L*	*mM/L*
A	Limestone (97.2% $CaCO_3$)	14.2	26.5	5.3
B	Dolomite (55.1% $CaCO_3$ and 44.3% $MgCO_3$)	363	689	.8
C	Magnesite (Wards)	405	792	1.5
D	Magnesite (Pennsylvania)	440	855	.5
E	Opal (Virgin Valley, Nev.)	325	369	778
F	do.[2]	246	296	949
G	Siliceous magnesium limestone (Calif.)	218	278	557
H	Pyrex glass cullet	120	175	747

[1] ASTM Method C 289–57T.
[2] Mix of 50 percent opal and 50 percent Ottawa sand.

active. In addition, mortar bars of this material were stored at 70° F. and failed to show significant expansion at 1 year (*11*). Since sample No. 34 is a borderline aggregate, the location of the solid line in the extreme lower portion of figure 1 may not be precise, as it is based upon the results of this aggregate alone.

The dashed line in figure 2 represents the criteria used by the U.S. Corps of Engineers (*10*), and here again there is good agreement between the position of both lines, except for the same type of spread in the lower part of the graph as noted in figure 1.

Single- versus double-end point titrations

An inspection was made of both graphs in order to determine whether the single- or double-end point titration procedures offered any distinct advantage. The U.S. Corps of Engineers apparently prefers the double-end point titration (*10*), while the Bureau of Reclamation and many other laboratories rely mainly on the single-end point titration. Table 3 lists the aggregates that failed to show good correlation between the chemical and mortar bar tests. This table shows that there is no difference between figures 1 and 2 in either the identification or number of such discrepancies. Therefore, no substantial advantage is evident for either type of titration procedure.

Comparison of Chemical and Mortar Bar Test Results

In some instances, the empirical classification of the aggregates into reactive and nonreactive materials, based upon the chemical test results, differs from the classification obtained by means of the mortar bar test. Specific consideration of samples showing such differences is of interest.

In figure 1, a total of 29 symbols appear to the left of the solid line. Of these, 24 are clear symbols denoting nonreactive aggregates and consequently are properly located in the graph. Three other aggregates in this area (Nos. 1, 35, and 42) were found to be of questionable reactivity, with mortar expansions of 0.08 percent at 1 year. However, the chemical test results of all three samples are located close to the dividing line where questionable aggregates might be expected.

Only two aggregates (Nos. 12 and 31), classified as either reactive or questionable, are located well to the left of the dividing line. There is no question as to the potential reactivity of sample No. 12, since this aggregate was found to cause significant mortar expansion even when combined with a medium alkali cement (*11*). However, both of these aggregates contain limestone or dolomitic constituents which, as discussed later, may invalidate the chemical test results.

A total of 23 symbols appear to the right of the solid line in figure 1. It would be expected that all aggregates located in this area should be reactive. However, seven samples were shown to be nonreactive by the mortar bar test. The identification of these aggregates is given in table 3. Sample Nos. 39, 50, and 51, included for information purposes only, may be eliminated from this discussion since they were not tested in mortar bars, but rather as concrete specimens prepared from gravel size aggregate.

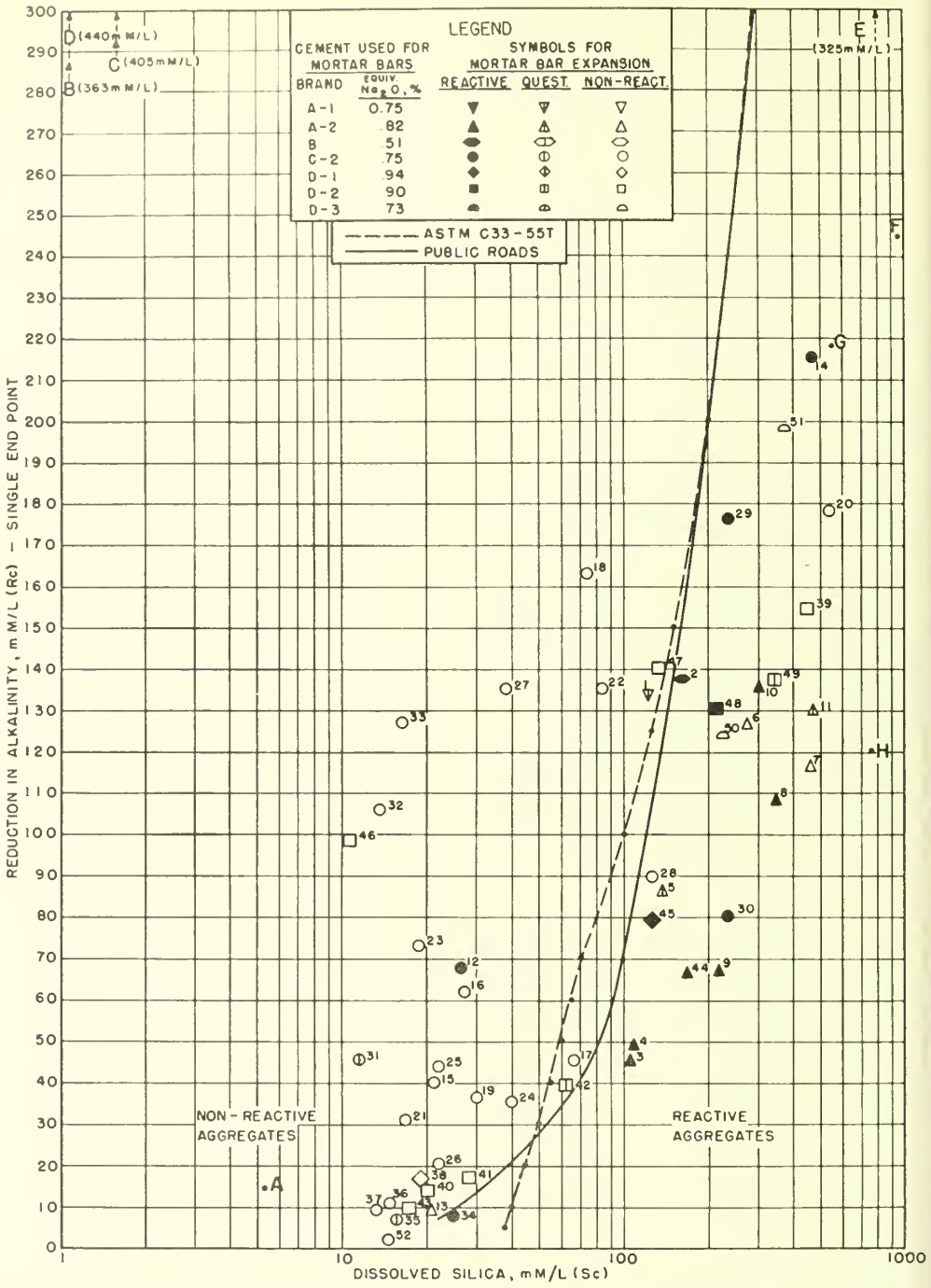

Figure 1.—Results of chemical test for reactivity of aggregate obtained by means of single-end point titration.

Therefore, only 4 samples (Nos. 6, 7, 20, and 28) are considered as limiting the degree of correlation between the chemical and mortar tests in this area. However, it was noted that all four samples exhibited some degree of mortar bar reactivity in supplementary tests. Sample Nos. 6, 7, and 20 caused considerable mortar expansion when the mortar contained an admixture of sodium hydroxide, while sample No. 28 began to show a questionable degree of reactivity in the mortar bar test at an age of 2 years (*11*). These latter findings lend support to the chemical test data obtained for these aggregates. It serves to emphasize that while some aggregates may be potentially reactive and are so characterized by the chemical test, these same aggregates may not be capable of producing dangerous expansion in the standard mortar bar test. Discrepancies of this sort are discussed more extensively in the following section.

Factors Affecting Correlation of Test Methods

Carbonate interference

The two aggregates (Nos. 12 and 31), which appeared as distinct discrepancies in the area to the left of the solid line in figure 1, were classified petrographically as containing certain carbonate constituents. Differential

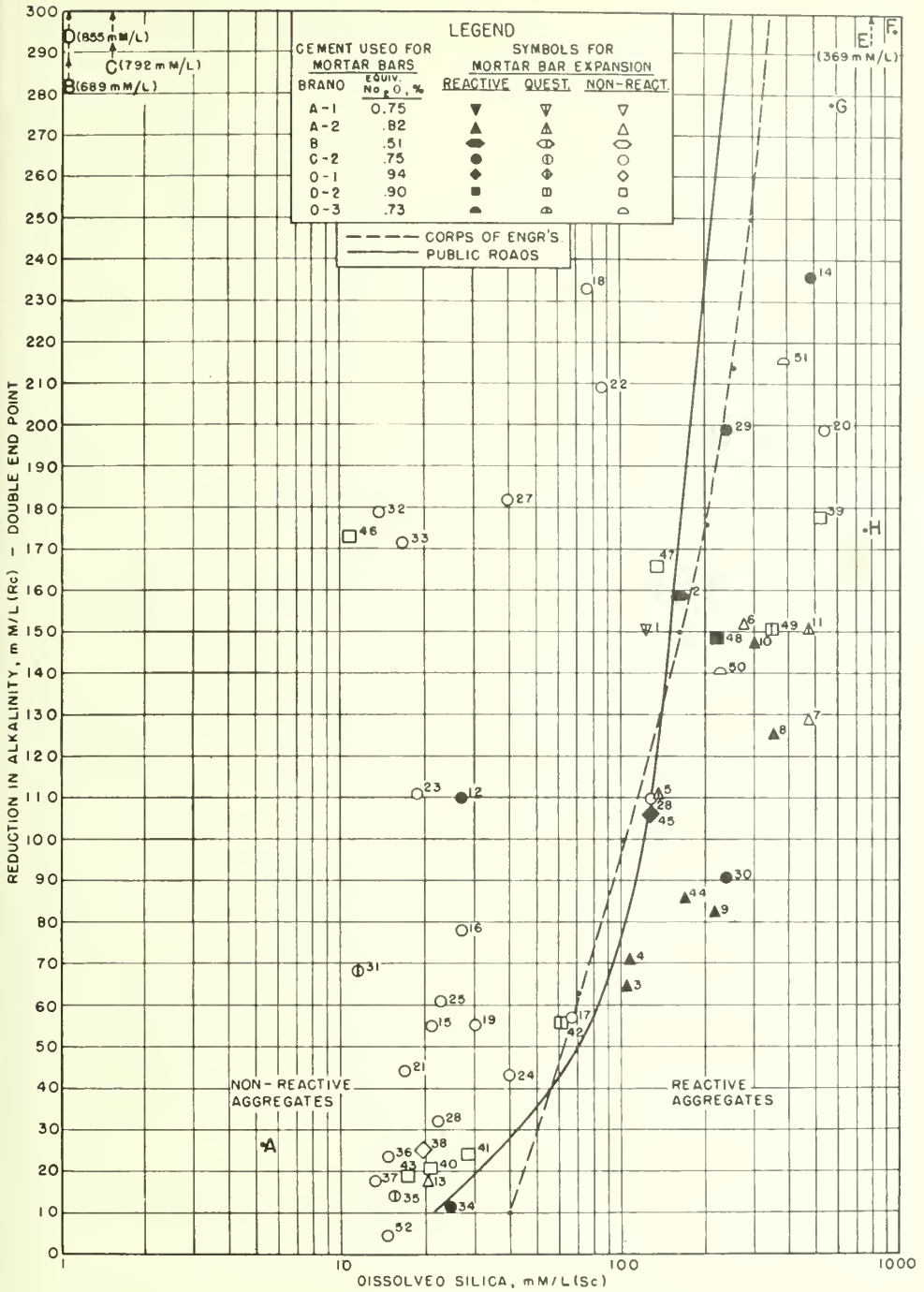

Figure 2.—Results of chemical test for reactivity of aggregate obtained by means of double-end point titration.

Table 3.—Identification of aggregates which showed poor correlation between chemical and mortar bar tests (selected by inspection of figures 1 and 2)

Position of plotted chemical test results	From figures 1 and 2	
	Aggregate identification	Number of samples
Left of solid line:		
Reactive aggregates [1]	No. 12 [2]	1
Questionable aggregates [1]	Nos. 1, 31, 35, 42	4
All aggregates		29
Right of solid line:		
Nonreactive aggregates [1]	Nos. 6, 7, 20, 28, *59, 50, 51* [3]	7
All aggregates		23

[1] Based on mortar bar test results and criteria shown in text and in table 1.
[2] Sample contains 16 percent dolomite as determined by DTA methods.
[3] Three samples (in italics) were not tested in mortar bars, but as concrete specimens prepared from gravel size aggregate.

thermal analysis showed that sample No. 12 contained both calcite and dolomite, while sample No. 31 contained essentially calcite without dolomite and magnesite. Mielenz and Benton (*9*), as well as Swenson (*6*), have indicated that dolomites and perhaps even limestones may at times cause significant interference in the chemical test. It was demonstrated that although certain opal-dolomite mixtures were found to be deleterious by mortar bar tests, the chemical test results gave false information concerning the reactivity of the mixture (*9*). Such misleading information was attributed to the interference of dolomite and other minerals such as magnesite, siderite, and serpentine. When present along with reactive constituents, these minerals produce greater reductions in alkalinity (Rc) and lower dissolved silica values (Sc) in the chemical test than would otherwise be obtained. The net effect of such interference is to shift the chemical test data both to the left and upward in figure 1, and thereby, at times, yield misleading information concerning a truly reactive aggregate.

A separate study of several carbonate materials was made in order to understand more fully their possible effects on the chemical test. Table 2 shows the chemical test results obtained on four carbonate minerals. Relatively pure limestone, dolomite, and two magnesites were tested, and the tabulated results were plotted in figures 1 and 2 as points A, B, C, and D.

As seen in table 2 and figure 1, the limestone (A) did not cause any significant alkalinity reduction, while the dolomite (B) and both magnesites (C and D) produced unusually large alkalinity reduction values (Rc). It would appear from these results, that pure limestone (calcium carbonate) does not react significantly with the sodium hydroxide solution used in the test, while dolomite and magnesite do. Furthermore, the ratio of the double- and single-end point values of Rc for both dolomite and magnesite, and the absolute difference between the values, suggest the formation of considerable quantities of weakly basic material in solution during the test.

The sodium hydroxide solution was analyzed in each case after the test was completed. No significant amounts of calcium or magnesium were detected nor was there any apparent decrease in the concentration of sodium below the level of the initial test solution. In the case of dolomite and magnesite, it was concluded that the test solution had lost some hydroxide ions in exchange for an equivalent amount of carbonate ions. The latter material accounts for the presence of a weak base after test. From these considerations, two possible equations can be written for the reaction between magnesite and the sodium hydroxide solution. Such a reaction may be represented by either of the following equations:

$$MgCO_3 + 2NaOH = Mg(OH)_2 + Na_2CO_3$$

$$5MgCO_3 + 2NaOH = 4MgCO_3 \cdot Mg(OH)_2 + Na_2CO_3$$

Such reactions appear reasonable from a consideration of the solubility products of magnesium carbonate (2.6×10^{-5} at 12° C.) and magnesium hydroxide (1.2×10^{-11} at 18° C.) (*12*). The latter material, a possible end product in the reaction, is much more insoluble in water than is magnesium carbonate, the reactant. The reaction between dolomite and the test solution undoubtedly is similar to that involving magnesite.

No special study was made to explain the depressed values of dissolved silica (Sc) caused by the presence of dolomite or similar materials (*9*). The following thoughts are offered as possible explanations of this effect: (a) The large reduction in alkalinity caused by such minerals may so reduce the alkalinity of the solution as to minimize its attack upon reactive forms of silica; (b) the formation of insoluble magnesium hydroxide or the basic carbonate may act as a protective film in limiting the attack of the solution upon reactive forms of silica; or (c) dolomite or its decomposition products may react with dissolved silica to form insoluble silicates and thereby remove silica from solution.

Elimination of carbonate interference

Efforts by Mielenz and Benton to eliminate carbonate interference in the chemical test by prior treatment of the aggregate with hydrochloric acid have not given satisfactory results (*9*). These authors have suggested that an analysis of the change in concentration of sodium and potassium rather than the reduction in alkalinity (Rc) would probably remove a large part of the spurious effects induced by such carbonates. While such a procedure would eliminate the falsely high Rc values, it would not eliminate the low Sc values produced by this interference.

Continued study of the "carbonate effect" on the chemical test is necessary in order to eliminate its interference. Because of such interference, a recent revision of the present ASTM chemical test recommends petrographic examination as a supplement to the chemical test data. This recommendation would involve a time-consuming petrographic examination of many aggregates, which on the basis of chemical tests would fall to the left of the dividing line in figure 1.

However, a more expedient alternative is suggested here. It has been shown that the "dolomite interference" is characterized by rather high Rc values in the chemical test. A level of Rc might be established so that all results below this level and to the left of the main dividing line may safely be considered nonreactive aggregates and, consequently, no further testing would be necessary. Results above this Rc level would require additional study by chemical, petrographic, X-ray, or differential thermal analysis (DTA) methods in order to detect the presence of either interfering or reactive minerals. The presence of small amounts of dolomite, magnesite, or calcite may be detected by spot tests or differential thermal analysis, while serpentine is conveniently identified by X-ray analysis. If the presence of such interfering minerals should be so detected, mortar bar tests would then be necessary in order to evaluate more fully the potential reactivity of the aggregate. The proper location of a dividing line is discussed later.

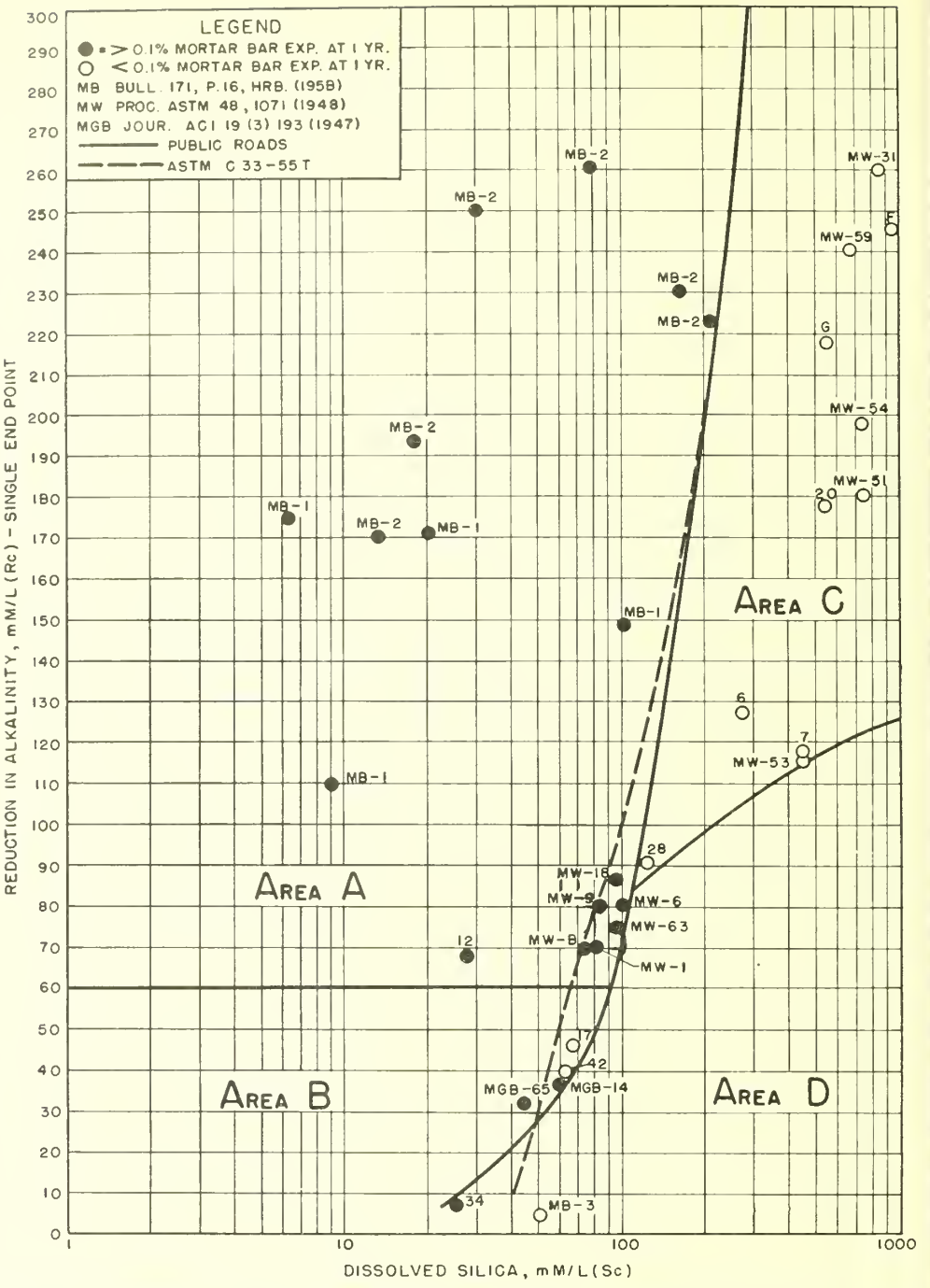

Figure 3.—Location of chemical test results that failed to correlate with mortar bar tests or other criteria.

Highly reactive materials

In addition to the results of this study, other investigators have found that many chemical test results falling to the right of the main dividing line fail to correlate with mortar bar test results (*4*, *7*, *8*). Both Sprague (*8*) and others (*7*) have attributed such discrepancies to the fact that certain aggregates contain such a large proportion of reactive constituents in relation to available cement alkalies that they fail to produce mortar bar expansion. This is a consequence of the "pessimum proportion"[3] of the aggregate being materially exceeded.

It is well known that certain opals and siliceous magnesium limestones are highly reactive and are so characterized by the chemical test. Yet when these materials are present in large proportions, they actually cause little or no expansion in the mortar bar test (*11*, *13*, *14*). It was shown by Woolf that 1 or 2 percent of opal combined with an inert material will produce dangerous mortar bar expansion, whereas much larger amounts of opal will cause insignificant expansion for a given cement alkali content (*16*).

Table 2 shows the chemical test results on

[3] Defined by T. E. Stanton (*13*) as that proportion of reactive constituent in an aggregate at which maximum mortar bar expansion occurs.

several well-known reactive materials; namely opal, siliceous magnesium limestone, and pyrex glass. These results are plotted in figures 1 and 2 as points E, F, G, and H. In figure 1, points F and G are particularly interesting since they approach the area where discrepancies (clear symbols) are shown. Point F represents a blend containing 50 percent opal and 50 percent Ottawa sand. McConnell and others have shown that a similar mixture failed to produce mortar bar expansion (*15*). Point G represents 100-percent siliceous magnesium limestone which Stanton has shown to cause negligible mortar bar expansion when tested in its entirety (*13*).

Division of reactive area

As a partial solution to the problem of such recurring discrepancies, Mielenz and Witte have suggested an arbitrary division of the area containing the aggregates classified as reactive by the chemical test (7). They suggested that such a division would be useful in interpreting the chemical test results. Their proposed division would separate the reactive area into an upper and lower portion. Chemically reactive aggregates which always caused mortar expansion were found to give chemical test results lying in the lower area, while chemically reactive aggregates which did not cause mortar expansion, when tested in their entirety, gave chemical test results in the upper area.

On the basis of the results obtained in this study, the proposed dividing line appears to have considerable merit. However, in order to separate the apparent discrepancies found in this study, such a dividing line should be located at a lower Rc level than that proposed by Mielenz and Witte. This study showed that all four of the aggregates having major discrepancies and located in the reactive area of figure 1 had Rc values of 90 or more. The data points for these aggregates should be located in the upper area of any arbitrary division.

Proposed Main Boundary Line and New Areas for Chemical Test

In order to establish acceptable area boundaries mentioned earlier, all discrepancies between chemical test and mortar test data were evaluated. Figure 3 shows the location of all such data points both from this study and the available literature. In addition, data related to the proper location of a main boundary line were similarly plotted. Results taken from the literature are partly identified in figure 3 by a letter symbol to denote the source.

Location of main boundary line

The data chosen for presentation with respect to the main boundary line are shown in the lower third of figure 3, and generally fall within the area between the two dividing lines.

The different positions of the two lines may be partially explained on the basis of the cements used in the mortar bar tests. The high-alkali cements in this study had an equivalent alkali content of less than 1.0 percent. The data given by Mielenz and colleagues

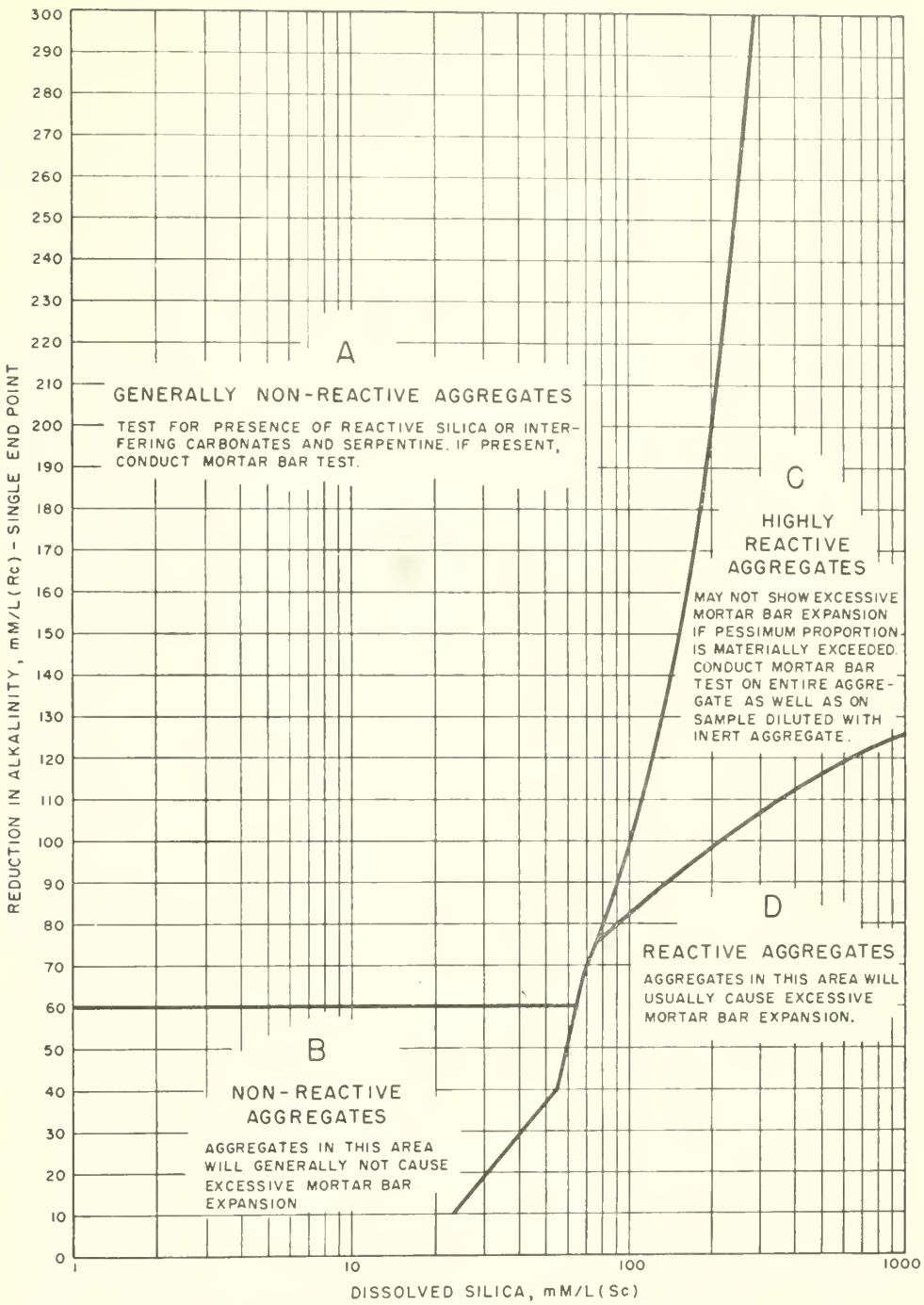
REDUCTION IN ALKALINITY, mM/L(Rc) - SINGLE END POINT
DISSOLVED SILICA, mM/L(Sc)
A
GENERALLY NON-REACTIVE AGGREGATES
TEST FOR PRESENCE OF REACTIVE SILICA OR INTERFERING CARBONATES AND SERPENTINE. IF PRESENT, CONDUCT MORTAR BAR TEST.
B
NON-REACTIVE AGGREGATES
AGGREGATES IN THIS AREA WILL GENERALLY NOT CAUSE EXCESSIVE MORTAR BAR EXPANSION
C
HIGHLY REACTIVE AGGREGATES
MAY NOT SHOW EXCESSIVE MORTAR BAR EXPANSION IF PESSIMUM PROPORTION IS MATERIALLY EXCEEDED. CONDUCT MORTAR BAR TEST ON ENTIRE AGGREGATE AS WELL AS ON SAMPLE DILUTED WITH INERT AGGREGATE.
D
REACTIVE AGGREGATES
AGGREGATES IN THIS AREA WILL USUALLY CAUSE EXCESSIVE MORTAR BAR EXPANSION.

Figure 4.—Evaluation of results of chemical test for potential reactivity of aggregates.

(*3*, 7), which provided the major basis for the ASTM line, were obtained with a cement of 1.38 percent equivalent alkali. Such differences in cement alkalies could cause a borderline aggregate to produce expansions less than 0.1 percent in one case and greater than 0.1 percent in the other. This, of course, would affect the position of any boundary line which is empirically drawn to separate reactive from nonreactive aggregates.

In order to establish a suitable boundary line for general use, the following considerations were taken into account: (1) The present ASTM line occupies a more conservative position with respect to the Bureau of Public Roads line in that there would be less danger of the nonreactive area containing chemical results of unsafe aggregates. (2) Samples MW–1, 8, 9, 18, 63, and MGB–14 were reported as causing deterioration in field concrete (*3*). Consequently, such aggregates should properly appear to the right of any dividing line. (3) Sample 42, in this study, showed 0.08 percent expansion at 1 year and could conceivably have given 0.1 percent expansion with a higher alkali cement.

In view of these considerations, the present ASTM line is preferable, except the lower portion should be adjusted to indicate clearly the deleterious nature of aggregates 34 and MGB–65. This can be accomplished by changing the slope of the lower portion of the ASTM line. For Rc values from 10 to 40, the formula for the adjusted line is as follows:

$$\log Sc = 1.2553 + 0.01213\ Rc$$

Figure 4 shows the proposed change in the

ASTM line between Rc=40 and Rc=10. All available published data were reviewed with respect to this change, and only one conflict was observed (aggregate MB–3). This aggregate, shown in figure 3, was a synthetic opal-quartz mixture with 0.1 percent opal which failed to show excessive mortar expansion (*9*). Its position is anomalous, even with respect to the original unchanged ASTM line, and no explanation of its position is attempted here.

Areas A and B

The left portion of figure 3 was evaluated, disregarding the aggregates just mentioned which had to do with the location of the main boundary line. It can be seen that all discrepancies (aggregates which produced mortar expansion) are in the upper portion of this area. No discrepancies appear below an Rc level of 60; therefore this value can be used to divide the left portion of figure 3 into two areas, A and B. In each case, the anomalous result shown in area A represents an aggregate which contains constituents which are known to interfere in the chemical test. Sample 12 contained dolomite; MB–1 contained various dilutions of opal in dolomite; and MB–2, various dilutions of opal in serpentine.

On the basis of the available data, area B should be clearly indicative of safe aggregates. Chemical results appearing in this area should be reliable and no further tests are required. In general, aggregates that appear in area A will also be safe; however, the presence of interfering minerals such as dolomite and serpentine may at times result in an unsafe aggregate appearing in area A. Consequently, all aggregates which fall into area A should be further evaluated for the presence of either reactive constituents or interfering minerals. In doubtful cases, mortar bar tests should be made.

Areas C and D

The right portion of figure 3 was similarly evaluated, again disregarding the results which had to do with the main boundary line location. All discrepancies appear in the upper portion of this area. Consequently, the right portion of the graph may be divided into two areas, C and D. No discrepancies appear in area D; therefore, chemical results which would appear in this area should be indicative of unsafe aggregates which are capable of producing excessive mortar expansion. No further tests on such aggregates would be necessary.

Area C contains numerous aggregates which did not produce mortar expansion (clear symbols). This area also contains unsafe aggregates as is evident in figure 1. Consequently, the purpose of area C is to focus attention upon aggregates requiring special investigation as a result of the chemical test data.

It is interesting to note that all aggregates in the upper portion of area C, figure 3, produce excessive mortar expansion when diluted to very small concentrations with inert material. Sample F is a 50–50 opal-Ottawa sand mixture; G and MW–59 are undiluted siliceous magnesium limestones from California; and MW–31 is an undiluted rhyolite tuff from Colorado.

On the other hand, several of the aggregates in the lower portion of area C, namely MW–51, 53, and 54, will cause excessive and maximum mortar bar expansion when present in about 20-percent concentrations with inert material (*7*). It may be that aggregates 6, 7, and 20 of the present study are similar and might have shown unsafe expansion upon dilution.

Consequently, chemical results which appear in area C suggest a highly reactive material, but do not necessarily preclude that all such materials will produce excessive mortar expansion. The ability to produce unsafe expansion will depend largely on the proportion of reactive constituent present. Therefore, all aggregates which yield chemical data in area C should be carefully examined by mortar bar tests. They should be tested in dilutions of 5, 10, 20, and greater percentages with inert materials, as well as in their entirety. This procedure is especially important where local shortages or special conditions require that aggregates from different sources be blended to make concrete.

Summary of New Areas and Boundary Line

Figure 4 indicates the significance of each of the proposed areas in evaluating chemical test results. It also shows the main ASTM boundary line, which has been modified at the lower end as discussed earlier.

The location of all boundary lines is, of course, empirical in nature and dependent exclusively upon available mortar bar and chemical test data. The location of such boundary lines may require minor adjustments as new data become available in the future. In addition, the limited reproducibility of the mortar bar and chemical tests, as reported by the U.S. Corps of Engineers (*10*, *17*), always raises some doubt as to the precise location of any dividing line. Consequently, marginal chemical test results, especially near the main boundary line, should be interpreted with some reservation.

Nevertheless, the present criteria shown in figure 4 should broaden the usefulness of the quick chemical test and, in many cases, eliminate the need for supplementing the chemical test with a long-term mortar bar test program or tedious petrographic analysis.

In one respect, the chemical test offers a unique advantage over the exclusive use of the mortar bar test. This applies to aggregates which appear in area C of figure 4. The engineer may obtain a false sense of security from a satisfactory (safe) mortar bar test result on such aggregates. However, it is possible that some of these materials will cause deterioration in concrete if blended with other relatively inert aggregates. The chemical test result would indicate the need for more careful evaluation of such materials.

(For conclusions, see pages 177 and 179.)

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(*Continued on page 190*)

Home-Interview Traffic Surveys and Related Research Activities

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported[1] by JOHN T. LYNCH, Chief, Planning Research Branch

IT HAS now been 15 years since the home-interview method of making origin and destination surveys in urban areas was developed. The first surveys by this method were made during World War II, when conditions were quite abnormal. The method has now been used in 126 urban areas and repeat surveys have been made or started in 10 of these. During 1958, surveys were begun in the following areas: Vallejo, Calif., Champaign-Urbana, Ill., Topeka, Kans., Minneapolis-St. Paul, Minn., Charlotte, N.C., Pittsburgh and Reading, Pa., and El Paso, Tex.

The Minneapolis-St. Paul, Charlotte, and Reading surveys were repeat studies. In addition, repeat studies were being organized for Denver, Philadelphia-Camden, and Nashville, and were under consideration for Atlanta, New Orleans, Tucson, and Honolulu.

The field methods and the information obtained have been changed very little since the earliest surveys. A few items have been added or altered, however, to aid in the study of trip projection. Present emphasis is on the improvement of analysis methods. It was recognized in the beginning that the most difficult problem was to find a satisfactory method of forecasting travel. It was thought that fundamental relations could be established which would permit the determination of the future travel pattern from projections of land use, population, and economic factors. For this reason information was obtained in the interview on such items as size of family, number of cars owned, occupation and industry, mode of travel, and purpose of trip.

Development of Land-Use Data

One important addition that has been made in the present practice is the determination of land use or the character of establishment, at the beginning and the end of each trip. This is done in considerable detail; for example, a shopping trip made to a food store is differentiated from one made to a clothing store, and a social trip made to a residence is recorded differently from a recreational trip made to a theater. Such detail requires about 60 land-use classifications. This detailed classification of land use at trip-ends was made first in the Chicago study, which is nearing completion, and is being made in most of the surveys that have been started during the past year.

The purpose of the detailed land-use information is twofold: First, the number of trips attracted to each class of development or establishment can be determined and related to some measurable factor such as population, automobile ownership, employment, dollar sales, or area; secondly, the effect of distance or travel time on trip attraction for each class can be studied. These are the important factors which enter into any interarea travel formula such as the so-called "gravity model."

Of course, it will not be practicable to forecast land use in any such detail. However, analysis of the data will aid in evaluating the variability of the trip attraction and distance factors for different land uses, in testing the form and possible accuracy of an interarea travel formula, in improving the zoning by grouping land uses of similar trip attraction characteristics to the extent practicable, and in estimating the factors for zones on the basis of approximate anticipated distribution of land uses within the zone.

Throughout the past 10 years, much research has been conducted on trip production and trip attraction in relation to land use. The results of many such studies have been published by the Bureau of Public Roads and the Highway Research Board (*1–8*).[2] Difficulties have been encountered because of mixed land uses within zones and lack of precise knowledge as to the character of the establishment at the ends of each trip. Progress has been slow because of the mass of detailed data to be analyzed and the shortage of personnel. However, some fairly reliable factors have been developed for trip production by purpose for residential zones, and for the effect of distance on shopping trips. What remains to be done is to test and evaluate various interarea travel formulas that have been proposed and to develop, if possible, statistically reliable factors for use in these formulas. Because of the availability of electronic computers, considerable progress is being made in this direction.

Interarea Travel Formula

The interarea travel formula for primary trips which is being evaluated at this time is, in general form, as follows:

$$T_{ij}=\left(P_i\frac{A_j}{\Sigma A}+P_j\frac{A_i}{\Sigma A}\right)\frac{K}{D_{ij}{}^n}$$

Where:

T_{ij}=The number of primary trips between zones i and j, that is, with one end at home.

P_i and P_j=The number of primary trips produced in zones i and j by residents of those zones.

A_i and A_j=The number of primary trips attracted to zones i and j by nonresidents plus intrazone trips by residents of those zones.

ΣA=The number of primary trips attracted to all zones.

D_{ij}=The distance between zones i and j, generally expressed in terms of travel time.

K=A constant and n an exponent, both to be determined empirically from the analysis.

After K and n have been calculated, the application of the formula will probably result in a different number of trips attracted to, and trips produced in each of the zones than was originally estimated from the land-use data. A series of successive approximations or some other method will then have to be used to make the number of trips in and out of each zone, calculated from the formula, agree with the original estimates of trip-ends.

This formula and a similar one for secondary, or non-home-based trips, are being evaluated by the Bureau of Public Roads with the aid of an electronic computer, using data from the Washington metropolitan area 1948 and 1955 O and D surveys. For trips produced and trips attracted (the P and A factors), actual survey values are used; the test is to determine whether a formula of this general form will produce reliable results, and if so, what values of K and n are applicable to different situations.

The test is being made by trip purpose, as it is expected that the value of the time exponent n will vary for different purposes. It is thought, for example, that travel time will be more of a deterrent for shopping trips than

[1] This paper was presented by Mr. Lynch before the Committee on Origin and Destination Surveys at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959.

[2] Italic numbers in parentheses refer to the list of references on p. 186.

for work trips. Also, it is expected that n will be smaller for trips to the central business district than for other trips.

In this test, the travel times reported in the interviews are being used. This part of the program has been completed and some preliminary runs have been made. The travel times appear to be reasonably consistent; the relation between peak hour and offpeak hour times, for example, are about as would be expected. It is necessary, however, to smooth out the reported times because of the small number of reported trips between some pairs of zones, and also because of the tendency to report times in even 5- or 10-minute intervals. This is being done through a study of time-distance relationships for pairs of zones similarly situated.

Data from the 1955 survey are being analyzed first. Subsequently, it is planned to use the data from the 1948 survey to see whether the values of K and n change over a period of time.

A similar test of this formula is being made by the Ohio Department of Highways, using data from the Dayton and Toledo surveys. These were controlled postcard surveys, and neither trip purpose nor travel time was obtained. It will be necessary, therefore, to confine the analysis to the total number of trips for all purposes, and to determine travel time from time runs. The survey will have special value, however, in permitting a comparison of the results for the two cities, as well as with those for Washington, D.C.

A program is being developed by the Bureau of Public Roads for testing the application of an interarea travel formula to travel between cities or counties. The States have supplied a large amount of data for this program from external surveys around urban areas throughout the country. Factors currently being considered are population, median income, automobile registrations, and persons employed in eating and drinking establishments and in hotels and other places of lodging. The last named factor is thought to be of special importance in connection with recreational travel. Trip production by residents and trip attraction for nonresidents will be considered separately as in the case of the formula that has been described. An adjustment of airline distance to highway distance will probably be used instead of travel time.

Electronic Computer Programing

Last year, Brokke and Mertz reported on a test of the Fratar, average-factor, and Detroit growth formulas for projecting the future urban traffic pattern from the present one (*9*). They found that the three formulas gave results of equal accuracy, but that the Fratar method required fewer iterations than the others. The standard errors for zone-to-zone movements were naturally high because of the very small number of trips between many pairs of zones. Zones were grouped, therefore, to give substantial trip volumes between each of the groups. When this was done, the standard errors were not unreasonably high. Since the grouped zones are much larger than would be used in practice, a different kind of test is thought to be necessary. What engineers actually wish to know is how accurate the results will be when a large number of zone-to-zone movements are accumulated on a freeway. It is planned, therefore, to accumulate the trips across grid lines and to determine the standard errors for sections of the grid crossed by substantial volumes of trips. This project is still in the planning stage, but it is hoped that it can be carried out during 1959.

A program to apply the Fratar formula has been written for a 705 IBM computer by the Bureau of Public Roads with the assistance of the National Bureau of Standards. This program has been used to project the future zonal traffic interchange from the 1955 data for the Washington area.

In the Chicago area transportation study, a program has been developed for the IBM 704 computer for assigning zone-to-zone traffic movements to a complete urban highway network, based on a minimum travel time concept (*10*). A similar program, based on somewhat different concepts, is being developed for the Detroit area traffic study.

A program to accomplish the same purpose has been written for the IBM 704 computer by the General Electric Computer Division for the Regional Highway Planning Committee of the Washington metropolitan area, with the assistance of Bureau of Public Roads personnel. This program, based on a time ratio rather than on a minimum travel time concept, will be described in a subsequent issue of PUBLIC ROADS magazine.

Programs similar to those mentioned undoubtedly can be developed for other computers of equivalent capacity and characteristics.

Measurement of Mass-Transit Usage

One of the most important accomplishments of 1958 was the developing and testing of what seems to be a satisfactory formula for estimating proportionate mass-transit usage. There has been a lack of knowledge in this field that would permit the distribution of future trips between mass transit and private automobiles with any degree of assurance. The formula involves a number of factors and is based on data obtained in part from the O and D surveys, in part from the transit companies, and in part from planning commissions. Full cooperation of such agencies in more than 20 cities has made the accomplishment possible.

The formula, as so far developed and tested, seems to predict the proportion of the trips made by mass transit in different cities with a surprising degree of accuracy. Preliminary tests indicate that it is also applicable to the prediction of mass-transit usage in different sections of the same city, but this will be further tested in 1959 as additional information becomes available.

In some of the O and D studies now being started, provision is made to obtain information on availability and cost of parking, and on walking time between place of parking or transit and origin or destination. It is thought that factors may be developed from such information that will improve the transit-usage formula.

Future Prospects

On the whole, the prospects for important research accomplishments during 1959 in the field of O and D studies seem bright. The number of people engaged in such research in the Bureau of Public Roads remains small, but the availability of electronic computers is speeding up the work. Furthermore, with greatly increased highway planning survey funds, the States are showing increased willingness and desire to engage in research that will aid them in forecasting traffic volumes and patterns. In the St. Paul-Minneapolis study, for example, $75,000 has been programed for research. In addition to the interarea travel formula test that has been mentioned, the Ohio Department of Highways is planning to undertake some research on the diversion, generation, and growth of traffic following the construction of rural freeway sections. Highway impact studies are underway in many States; in some cases the work is being done by State personnel and in others, by university staffs. The North Carolina State Highway Commission is conducting several traffic research studies, through their highway planning division and through university cooperation.

In most of the current urban O and D studies, continuing organizations are being set up or planned. These organizations will, in many cases, carry out research. In Detroit, for example, continuing research is being planned through an agreement with Wayne University.

Considering all of these factors, it is reasonable to hope that the analysis and projection of traffic patterns will be placed on a sound scientific basis within a few years.

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Travel to commercial centers of the Wash-

(Continued on page 190)

Pretreatment of Soils and Clays for Measuring External Surface Area by Glycerol Retention

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported by EARL B. KINTER and SIDNEY DIAMOND, Highway Physical Research Engineers

Many soil properties important in highway work, such as plasticity and volume change, are caused by the interaction of soil particle surfaces and water. Similarly, interactions between particle surfaces and chemical additives are involved in soil stabilization. In a given soil, both types of interaction are largely controlled by the amount of surface area possessed by the soil particles.

A convenient and accurate method for measuring the surface area of soil materials should therefore be quite useful in studying and evaluating soil properties for highway purposes. Such a method was developed by the Bureau of Public Roads in 1956. The present article reports results of more recent work concerning a refinement of the original test procedure.

IN TWO previous reports by the authors (*1*),[1] a method was presented for determining the surface area of clays and soils from the weight of sorbed glycerol retained by specimens after extended heating at 110° C. in the presence of free glycerol vapor. For the particles of clay minerals such as kaolinite, only external surfaces are available for sorption; but for expanding minerals such as montmorillonite, the glycerol is also retained on internal surfaces between the unit layers of the crystallites.

This internally sorbed glycerol forms what has been called a "monointerlayer" complex, that is, a monomolecular layer or film between two planar clay surfaces. Each molecule of glycerol in such a complex covers twice as much clay surface as does a molecule sorbed on external surfaces. The factor for conversion of a given weight of the internal film to values of surface area is therefore twice as large as for an equal weight of a similar film on external surfaces. For the general use of the glycerol retention method with soils and clays, it is thus necessary to evaluate separately the glycerol retained on the two types of surface, unless it is known that even trace quantities of the expanding minerals are not present.

Procedure

The procedure employed for this purpose was adapted from that used by Dyal and Hendricks (*2*) in connection with the sorption of ethylene glycol by clays. As applied to the sorption of glycerol, this procedure consisted of determining separately the glycerol retained by an untreated sample and by a sample of the same material previously heated to 600° C.

It was assumed that the heat treatment would irreversibly collapse expanding mineral lattices so as to seal the internal surfaces against subsequent penetration by glycerol and permit sorption on only the external surfaces; and since the untreated material would retain glycerol on both types of surface, the difference in retention between the two samples was accordingly attributed to internal surfaces.

For this procedure to result in a realistic measure of the surface area of a given sample, it is obvious that the heat treatment must completely and irreversibly collapse any expanding layers present, but must not, at the same time, appreciably alter the original external surfaces of constituent particles. However, depending on the composition of the sample to be studied, one or both of these conditions may not be fulfilled. For example, in the first report cited (*1*), the authors showed that with certain montmorillonites the 600° C. heat treatment is not sufficient for irreversible collapse. These montmorillonites would, therefore, still sorb glycerol internally to some extent, and thus the amount attributed to external surfaces would be too high. On the other hand, temperatures above 600° C. cannot be used to ensure irreversible collapse because evidence exists that this would sometimes severely reduce external surface areas (*3*).

The heat treatment at 600° C. may also affect minerals other than montmorillonite. Nitrogen adsorption measurements (*4*) have shown that the surface area of kaolinite was reduced by about 20 percent on heating to this temperature. The present authors had noted that a substantial reduction in glycerol retention is also produced with illite, but with this mineral the reduction was ascribed to the irreversible collapse of expandable layers contained within the illite particles. The external surface of vermiculite, on the other hand, has been shown to increase threefold on heating to 600° C. (*5*).

It is also to be expected that heating would result in pronounced increases in surface area with certain of the common accessory "nonclay" minerals of soils and clays. For example, Russell and Cochran (*6*) observed that the surface area of an aluminum trihydrate closely similar to natural gibbsite increased from about 10 to 425 square meters per gram ($m.^2/g.$) on heating to 400° C.; and a subsequent decrease to about 280 $m.^2/g.$ occurred on heating to 600° C. Similar increases should be anticipated for the crystallized hydrous iron oxides, goethite, and lepidocrocite, although conversely, heating an amorphous ferric oxide gel to 600° C. has been shown to reduce the specific surface area from about 250 to 50 $m.^2/g.$ (*7*).

In general, then, the value of 600° C. heating is doubtful at best for materials of such varied composition as soils and clays. In the search for a suitable substitute, it was thought that a chemical pretreatment might be found by which the entrance of glycerol molecules into the interlayer spaces of expanding minerals could be prevented, without either interfering with normal adsorption on external surfaces or reducing or increasing the external surface area of any of the minerals present.

Since exchangeable cations are known to be distributed between the unit layers of montmorillonite, attention was first given to cation exchange treatments. None was found suitable, however, until, as suggested by the work of Greene-Kelly (*8*), the triethylammonium cation (TEA) was tried. Greene-Kelly had reported that a montmorillonite saturated with TEA exhibited a consistent basal spacing of approximately 13.3 angstroms (1 A.=one hundred-millionth of a centimeter), regardless of whether the sample was air-dried, immersed in water, or immersed in glycerol. In trials with several other montmorillonites, the present authors confirmed these observations and also noted that the 13.3A. spacing was not altered when specimens were dried at 110° C. or washed with water or acetone.

This apparently very stable spacing indicates a distance of about 3.8A. between the montmorillonite layers, which would naturally exclude glycerol molecules since they

[1] Italic numbers in parentheses refer to the list of references on page 190.

are about 4.5A. in diameter. Preliminary trials with montmorillonites indicated that saturation with TEA definitely blocked the penetration of glycerol into the interlayer spaces, and apparently did not interfere with normal adsorption on external surfaces. The investigation of the use of TEA saturation as a replacement for the 600° C. heat treatment forms the basis of the present report.

Conclusions

The evidence presented indicates that when soils and clays are heated at 600° C. as a pretreatment for the collapse of montmorillonite in the determination of external surface area by glycerol retention, the external surface area of several other associated minerals is seriously affected and a complete collapse of montmorillonite is not necessarily obtained. This pretreatment accordingly may introduce large errors in surface area measurements, and is therefore unsuitable.

On the other hand, the data show that pretreatment by saturation with the triethylammonium cation effectively prevents the adsorption of glycerol on the internal surfaces of expanding clay minerals, but does not alter external surfaces nor otherwise interfere with their normal adsorption of glycerol. The triethylammonium saturation thus permits accurate measurements of surface area, and is therefore suitable for general use with soils and clays.

Materials Studied

The clays and soils used in the study are described in table 1. They were selected to provide representatives of the principal silicate clay minerals, and also one of the common hydrous oxides.

A normal solution of triethylammonium chloride, used for saturating the soils and clays, was prepared by neutralizing triethylamine (Eastman) with hydrochloric acid to pH 7.0, and diluting with distilled water.

The retention of glycerol by the experimental materials was determined essentially as described previously (*1*). Briefly, the procedure as used here involved preparing duplicate 1- to 3-gram specimens of material passing the No. 40 sieve; drying at 110° C. in aluminum foil dishes and weighing to 0.2 mg.; adding 10 ml. of a 2-percent aqueous solution of glycerol; and heating to constant weight at 110° C. in a mechanical convection oven containing a constant supply of glycerol vapor at a partial pressure just below saturation. The gain in weight represents the glycerol retained, and is expressed as a percentage of the 110° C. oven-dry weight.

Three determinations of retention were made for each material: (1) untreated, (2) previously heated to 600° C. for 4 hours, and (3) previously saturated with TEA. Saturation was accomplished by treating 10 grams of material with an amount of the triethylamine chloride solution sufficient to exceed the cation exchange capacity fivefold. The material and solution were thoroughly mixed, centrifuged, and the supernatant liquid decanted. This treatment produced strong flocculation with all of the materials studied, and gave very clear supernatant liquids with a minimum of centrifugation. After three such treatments, the material was thoroughly washed in the same manner with acetone, dried over a steam bath, and lightly pulverized.

Apparent values of external surface were calculated from the glycerol retained after either pretreatment by multiplying the percentage retained by the factor 17.65; the apparent internal surface in either case was computed as 35.3 times the difference between the percentage of glycerol retained by the untreated sample and the percentage retained by the treated sample (*1*).

Since all calculations were made on the basis of the 110° C. dry weight, specimen weights of the 600° C. heated material were corrected for the loss in weight between 110° C. and 600° C. A similar correction of the TEA saturated material to the same basis would involve adjustment for the increase in weight due to the excess weight of the triethylammonium cation over that of the cation it replaced. This correction was applied only to the montmorillonite samples, since it was considered negligible for the others due to their considerably lower cation exchange capacities.

Test Results

Data for a comparison of the effects of the two pretreatments are given in table 1, including experimental values of glycerol retention and the corresponding calculated values of external surface area. For two of the clays, surface area values obtained by the Brunauer-Emmett-Teller (BET) method using nitrogen sorption are also included. In the following discussion, the experimental materials are grouped according to mineral composition.

Montmorillonite

The external surface area obtained for Southern bentonite after 600° C. heating agrees quite closely with that after TEA saturation. This implies that with this montmorillonite, neither pretreatment significantly alters the mineral surfaces or interferes with normal adsorption, and that both effectively prevent the entrance of glycerol molecules into interlayer spaces. These implications are substantiated by the good agreement between the results for the glycerol sorption and the BET methods. The area indicated for the latter method was determined by Lopez-Gonzalez and Deitz (*3*) on another specimen of the sample used in the present work.

With the Wyoming bentonite, on the other hand, it was necessary to heat to 700° C. to irreversibly collapse the lattice, and the external surface area found after heating to this temperature is about 5 m.²/g. lower than that after TEA saturation. If this is a significant difference, it is probably due to a slight sintering effect of the heat treatment at the higher temperature.

Kaolinite

Table 1 includes data for two kaolins. With the Edgar Plastic Kaolin (E.P.K.), TEA saturation resulted in a glycerol retention slightly lower than that of the untreated material. The reduction is thought to be due to a small quantity of montmorillonite,

Table 1.—Effect of pretreatments on external surface area

Sample No.	Material	Clay mineral type	Source	Glycerol retention [1]			External surface area		
							Calculated from glycerol retention after—		From nitrogen adsorption, BET method
				110° C. dried	600° C. heated	TEA saturated	600° C. heating	TEA saturation	
				Pct.	*Pct.*	*Pct.*	*m.²/g.*	*m.²/g.*	*m.²/g.*
R-145	Southern bentonite [2]	Montmorillonite	Panther Creek, Miss.	20.7	4.55	4.61	80.3	81.4	[2] 83.8
R-51	"Volclay" Wyoming bentonite	do	American Colloid Co.	19.0	[3] 2.56	2.85	[3] 45.2	50.3	
R-164	Edgar plastic kaolin (E.P.K.) [4]	Kaolinite	Edgar Clay Co.	2.49	1.70	2.20	30.0	38.8	[4] 35.6
R-165	Lustra kaolin [4]	do	Southern Clay Co.	.81	.72	.85	12.7	15.0	
R-152	Fithian illite	Illite	Fithian, Ill.	6.1	3.5	4.5	62	79	
R-170	"Mountain leather" [5]	Palygorskite	Metalline Falls, Wash.	9.5	19.0	9.3	335	165	
S-61	Ando soil [6]	Allophane	Japan	16.2	8.3	16.2	146	293	
PC-1110	Grizzly soil	Allophane, montmorillonite	B horizon; Crook County, Oreg.	13.0	4.4	7.0	78	124	
PC 246	Linker soil	Gibbsite, kaolinite, soil chlorite	B horizon; De Kalb County, Ala.	1.75	2.2	1.4	37	25	

[1] Each glycerol retention percentage is an average of values from duplicate specimens.
[2] Sample and nitrogen surface area value supplied through the courtesy of V. R. Deitz, National Bureau of Standards.
[3] Sample heated to 700° C., since this temperature was required to secure irreversible lattice collapse.
[4] Sample and nitrogen surface area supplied through the courtesy of W. C. Ormsby, National Bureau of Standards.
[5] Sample supplied through the courtesy of E. B. Henderson, U.S. National Museum.
[6] Sample supplied through the courtesy of L. D. Whittig, formerly of the U. S. Department of Agriculture.

amounting to about 1 percent. This mineral could not be identified in an X-ray diffraction examination of the whole material, but was definitely detected in a very fine fraction separated by centrifugation. On the other hand, a further and substantial reduction in glycerol retention is indicated for the 600° C. heat treatment. This is thought to be the result of a lowering of the surface area by the heat treatment itself, in accordance with the previously cited experience of Gregg and Stephens (*4*). It is also noteworthy that surface area values obtained from the glycerol retention are in good agreement with those given by the BET method for a sample from the same stock.

With the Lustra kaolin, TEA saturation did not result in a reduction in glycerol retention, which indicates that this material is virtually free of montmorillonite. The 600° C. pretreatment, however, produced a modest decrease in retention and surface area, again in agreement with the findings of Gregg and Stephens.

Illite

Saturation of Fithian illite with TEA reduced glycerol retention from 6.1 to 4.5 percent, which is interpreted as due to a collapse of expandable layers equivalent to a content of about 7 percent of interlayered montmorillonite. The indicated value of external surface area, 79 m.²/g., is in good agreement with values quoted by Brooks (*9*).

On the other hand, when the 600° C. heat treatment was used, retention was 3.5 percent, which corresponds to an external surface of 62 m.²/g. This effect closely parallels the findings of Orchiston (*10*), who noted a reduction from 78 to 57 m.²/g. on heating Fithian illite to this same temperature.

Palygorskite

The sample of "mountain leather" consists chiefly of palygorskite, along with a small content of quartz. No montmorillonite could be identified by X-ray diffraction, in contrast to the more familiar "Georgia attapulgite" which the authors have found to contain a considerable amount. The absence of montmorillonite from the present sample is further attested to by the fact that there is essentially no difference between the glycerol retention values of the untreated and the TEA saturated material.

In contrast to the negligible effect of the TEA saturation, the glycerol retention after 600° C. heating was more than double that of the untreated material. This indicates that the heat treatment produces a large amount of new surface area. This apparently contradicts the results obtained by McCarter et al. (*11*) with nitrogen adsorption measurements, which indicated no change in the surface area of the attapulgite form of this mineral on heating to this temperature. However, the results for the two adsorbates can be reconciled if the difference in molecular size is taken into account. These authors state that the dimensions of the structural channels which run parallel to the long axis of the fibrous particles are such that the inner surfaces are already accessible to the nitrogen molecules in the unheated material. Therefore, the heating could not expose any additional surface of this type to nitrogen adsorption.

On the other hand, it is thought that glycerol molecules are too large to enter the channels of the palygorskite before it is heated, but the large increase in retention after heating suggests that they can then do so.

It is of interest to note that the external surface area, calculated from the glycerol retention of the TEA saturated palygorskite, corresponds approximately to what would be expected from the dimensions of the individual fibers of attapulgite reported in the literature. The value of 165 m.²/g. given in table 1 corresponds to an average fiber diameter of about 200 A.; examination of attapulgite in the electron microscope (*12*) has indicated that the fibers are about 50 to 100 A. thick and about 100 to 300 A. wide.

Allophane

An Ando soil from Japan was selected for studying the surface area of allophane. This soil was shown by X-ray diffraction to consist of noncrystalline material, except for a small content of quartz and feldspar. Its differential thermal analysis curve was identical to that of "allophane A" as described by Fieldes (*13*). The glycerol retention of the untreated material is almost as high as that of montmorillonites. Nevertheless, the fact that the retention is unchanged by TEA saturation is considered as further evidence that montmorillonite is not present, and that all of the surface area is external.

In direct contrast, the 600° C. pretreatment resulted in a reduction of retention to approximately half that of untreated or TEA saturated specimens. This drastic effect is thought to be due to sintering, but regardless of its cause, it clearly indicates that for soils and clays containing allophane, pretreatment at 600° C. would lead to very large errors in surface area.

Allophane-montmorillonite

The Grizzly soil was selected for study because X-ray diffraction indicated that it contains a large amount of amorphous material, along with an appreciable admixture of montmorillonite. Since this soil is the weathering product of materials of volcanic origin, the amorphous constituent is assumed to be allophane. Differential thermal analysis supported this interpretation and gave no indication of the presence of free hydrous oxides of iron or aluminum.

The glycerol retention data given in table 1 are what would be expected for a material containing montmorillonite and allophane. The retention value of the untreated material is reduced from 13.0 to 7.0 percent by the effect of the TEA saturation on montmorillonite; however, with heat treatment alone a retention of 4.4 percent was obtained, which indicates some destruction of the surface area of the allophane constituent as a result of the heating. Accordingly, in estimating the external surface area of this soil, an apparently very low value of 78 m.²/g. is obtained after the heat treatment, whereas a more realistic value of 124 m.²/g. is given by TEA saturation.

The difference in retention of the untreated and the TEA saturated soil indicates a montmorillonite content of about 25 percent. This is substantially in accordance with the X-ray diffraction evidence.

Hydrous oxides in soils

The clay fraction of Linker soil, which was included to represent materials containing hydrous oxides, was shown by X-ray diffraction and different thermal analysis to consist largely of gibbsite, along with kaolinite and a soil chlorite as accessory minerals. A small reduction in the glycerol retention is noted for TEA saturation. If this is interpreted as due to internal surface, it corresponds to a little over 1 percent of expandable layers, presumably contained as mixed-layers in the soil chlorite.

In contrast to most of the other materials studied, the Linker soil was found to increase in retention as a result of the 600° C. treatment. This can be attributed to an increase in surface area from the decomposition of the gibbsite, in accordance with the previously cited work of Russell and Cochran (*6*).

An analagous effect was noted for a relatively pure mineralogical sample of lepidocrocite, which showed an increase in retention from 0.3 percent for the untreated material to 3.0 percent for specimens previously heated to 450° C. A sufficient quantity of the lepidocrocite was not available for a thorough study,

Table 2.—Effect of pretreatments on internal and total surface areas calculated from glycerol retention

Sample No.	Material	Clay mineral type	Internal surface area, calculated from retention after—		Total surface area, calculated from retention after—	
			600° C. heating	TEA saturation	600° C. heating	TEA saturation
			m.²/g.	*m.²/g.*	*m.²/g.*	*m.²/g.*
R-145	Southern bentonite	Montmorillonite	570	568	650	649
R-51	Wyoming bentonite	do	580	570	625	620
R-164	Edgar plastic kaolin	Kaolinite	28	10.2	58	49
R-165	Lustra kaolin	do	3.2	0	15.9	15.0
R-152	Fithian illite	Illite	92	56	154	135
R-170	"Mountain leather"	Palygorskite	0	0	335	165
S-61	Ando soil	Allophane	278	0	424	293
PC-1110	Grizzly soil	Allophane, montmorillonite	304	212	382	336
PC-246	Linker soil	Gibbsite, kaolinite, soil chlorite	0	12.3	37	37

but the data for both of these materials indicate that the 600° C. heat treatment is clearly unsuitable for soils and clays containing the crystalline hydrous oxides of iron or aluminum.

Effect of Pretreatment on Estimation of Internal and Total Surface Areas

In addition to the effects of the 600° C. heat treatment on external surface areas, there are corresponding errors in the calculations of internal and total surface areas. Values of internal and total surface areas for both the 600° C. heating and the TEA saturation treatments were calculated from the data in table 1, and are presented in table 2. Internal surface areas were calculated by the method given earlier, and the total area is, of course, the sum of the internal and external areas.

From the data given in table 2, it is seen that for those samples having internal surface, the internal surface areas for the 600° C. treatment vary quite widely from those for TEA saturation, except for the montmorillonite samples. In estimating the total surface area, on the other hand, the method of calculation itself provides a compensating mechanism, so that differences for the two pretreatments are not large, except for the materials containing palygorskite or allophane.

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Nonelastic Deformations in Continuous Concrete Structures

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The performance of continuous-reinforced concrete bridges has been rather unsatisfactory because of the objectionable cracks and sags that develop in the spans following construction. This difficulty can be alleviated, however, by modifying the designs to compensate for the effect of the drying shrinkage of concrete. Creep and plastic flow of the concrete are also recognized as minor contributing factors to the problem of cracking and sagging.

Designing for shrinkage moments usually adds less than 1 percent to the cost of a structure. Heavier and longer lengths of negative moment reinforcing steel are required because of the increase in negative moments and because the points of inflection move farther out in the spans as a result of the increase in negative moments produced by the effect of drying shrinkage of the concrete. For some designs the concrete on the compression side of the girders will be overstressed in the vicinity of the interior piers. A slight change in the geometry of the girders usually reduces these stresses sufficiently and adds only a very small volume of concrete to the design. In box girders this can be accomplished by thickening the compression flange of each girder for a short distance at each interior pier.

CONTINUOUS-REINFORCED concrete bridge spans when designed by the elastic theory with the reinforcing steel stressed to 20,000 p.s.i. and the concrete stressed to 1,200 p.s.i. by combined dead load, live load, and impact moments, as permitted by the American Association of State Highway Officials' *Standard Specifications for Highway Bridges*, usually continue to deflect under dead load for several months after completion. This nonelastic deflection is accompanied by severe cracking of the floor slabs in the negative moment lengths of the girders. Some spans may sag sufficiently to impair the riding quality of the roadway surface and mar the appearance of the structure. Careful observance of continuous-reinforced concrete deck girder, I-girder, and box girder spans has shown that these defects may appear as early as 3 months after completion of the work. Some spans develop objectionable sag before highway live loads are placed on the structure.

Studies made to determine the cause of this behavior revealed that reinforced concrete simple spans continue to deflect for about 4 years after construction. The final deflection may become as much as four times the elastic deflection under dead load at the time of centering removal. In long-span continuous-reinforced concrete deck girders, I-girders, and box girders designed for maximum allowable unit stresses under dead load, live load, and impact moments, the nonelastic deflection is accompanied by numerous cracks across the floor slabs in the negative moment areas of the girders. In the three-span continuous layouts with end spans less than 75 feet, most of the sag occurs in the middle span. In the longer spans appreciable sag occurs in all members. Some examples of cracking and sagging found in existing three-span bridge systems follow.

Measurements of Sags and Cracks in Completed Structures

A continuous box girder with 100-120-100-foot spans sagged 2 inches in the center span and 2 1/16 inches in the end spans after the bridge was opened to traffic. A 75-100-75-foot span continuous I-girder had 5/8- and 7/16-inch sags in the end spans and 1/2 inch in the center span below theoretical finished grade, 2 1/2 years after completion. These spans had approximately 1/2-inch camber when completed. A 50-70-50-foot span continuous-reinforced concrete deck girder was about 8 months old before any live loads were placed on it. These spans cracked, and the middle span sagged 1/2 inch under dead load when only 5 to 6 months old. Two identical continuous box girders with 75-100-75-foot spans were built to maximum camber ordinates of 1 inch in the end spans and 1 3/8 inches in the middle span. Approximately 1/2-inch camber remained when the structures were completed. Levels taken when the spans were 3 years old indicated 3/8- and 1/4-inch sags in the end spans and a 3/4-inch sag in the middle span, measured below theoretical finished grade.

One of the latter two bridges had typical cracks and sags. The cracks over the center of the interior piers and for about 5 feet on either side were on approximately 12-inch centers, their spacing increasing to about 2-foot centers, 20 feet on either side of the piers. A tool was made for use with a micrometer to measure the width of the cracks. The width of those on 12-inch centers varied very little. The cracks directly over the center of piers measured 0.0312 inch. The maximum elastic elongation of reinforcing steel in a 12-inch length equals $(32{,}000 \times 12)/30{,}000{,}000 = 0.0128$ inch. Assuming shrinkage of 0.0006 inch per inch in the concrete—neglecting the probability that the concrete carries some tension and disregarding the effect of creep which tends to nullify the shortening effect of shrinkage—the concrete will shorten $0.0006 \times 12 = 0.0072$ inch in 12 inches. Then the maximum width of the cracks for these extreme and very improbable conditions should be 0.0200 inch.

In view of tension in the concrete and the creep effect, the concrete fiber shortening factor is probably less than 0.0003 inch over the center of piers. Then the maximum width of the cracks in concrete with reinforcing steel stressed to the elastic limit should not exceed 0.0164 inch. Under the theoretical dead load stress in reinforcing steel of about 12,000 p.s.i., the width of the cracks should not exceed 0.0088 inch. Therefore, the cracks over the center of piers measuring 0.0312 inch were nearly four times larger than they should have been under the theoretical existing stress ($0.0312 - 0.0088 = 0.0224$ inch). The widths of the cracks indicated that the negative moment reinforcing steel had stressed beyond the elastic limit.

One continuous-reinforced concrete deck girder bridge with 60-70-60-foot spans was found which had no perceptible sag in any span and there were no cracks in the floor slabs. This bridge had been in place for about 4 years. The spans were carefully analyzed and it was found that the theoretical stresses in the reinforcing steel and the concrete were very low in the parts of the girders subject to negative moment.

The data developed from these investigations indicated clearly that the dead load negative moments were increasing with time and that the rate of increase paralleled known rates of increase in drying shrinkage of aging concrete. It was concluded that the increase in dead load negative moments was produced by a gradual change in the physical shape of the continuous system, primarily through the effect of drying shrinkage in the concrete. It was recognized that creep and plastic flow could be minor contributing factors. The study indicated that perceptible sag developed in spans only when the negative moment reinforcing steel was stressed beyond the elastic limit or when the concrete was stressed into the plastic flow range.

Modification of Bridge Design

A method of determining the probable mechanical effect of drying shrinkage of concrete in continuous concrete bridge spans was developed by the author in 1955. The new theory is explained in subsequent sections of

this article. The studies leading to the development of the theory were begun in 1952 after engineers of the Alabama State Highway Department and the Bureau of Public Roads became concerned with the safety of the 100-120-100-foot continuous box girder spans mentioned earlier and other continuous concrete spans. The State Bridge Engineer was considering abandoning this type of design on future State and Federal-aid bridge projects unless a solution to this problem could be found.

After observing sags and cracks in the spans of two structures—an 80-100-80-foot span continuous concrete I-girder and a 50-70-50-foot span reinforced concrete deck girder—the designs were modified by the State highway department to agree with the theory of increased dead load negative moments due to drying shrinkage in the concrete. Two I-girder type bridges of the modified design were completed on Federal-aid projects early in 1956. The girders, in the negative moment lengths, were designed for combined dead load, live load, impact, and shrinkage moments using a shrinkage factor of 0.0006. Levels taken periodically at one of the bridges disclose no measurable increase in dead load deflections since removal of the falsework, nor have cracks been observed in the negative moment lengths of either set of spans. Another bridge of the deck girder type, similar to the one that had cracked and sagged under dead load, has been built using the modified design. After being in use 3 years, no cracks or sags had developed.

Twelve existing designs have been modified and fifty new three- and four-span reinforced concrete deck girder and I-girder standard designs have been prepared by the State highway department for use on Interstate System grade separation structures and other State and Federal-aid bridge projects. Several continuous box girder designs for use on Interstate System grade separation structures have been prepared by a consultant. All girders, in the negative moment lengths, were designed for combined dead load, live load, impact, and shrinkage moments using a factor of 0.0006. Unit stresses for the combined moments do not exceed the allowable unit stresses permitted by the AASHO specifications. Twelve Interstate System grade separation structures and several bridges over streams, constructed according to the modified designs, have been completed for several months; none of the spans have shown a tendency to crack or sag.

Designing for shrinkage moments adds very little to the cost of a structure, usually less than 1 percent. The quantity of negative moment reinforcing steel is increased some due to the additional requirements caused by the increase in negative moments and to the additional lengths needed because the points of inflection move further out in the spans. In some designs the concrete in the compression flange will be overstressed over a short length of the girders in the immediate vicinity of the interior pier supports. Modifying the geometry of the girders throughout these lengths to reduce the concrete stresses usually requires a very small amount of additional concrete. An extra short compression reinforcing bar or two will reduce the concrete stresses sufficiently in some designs.

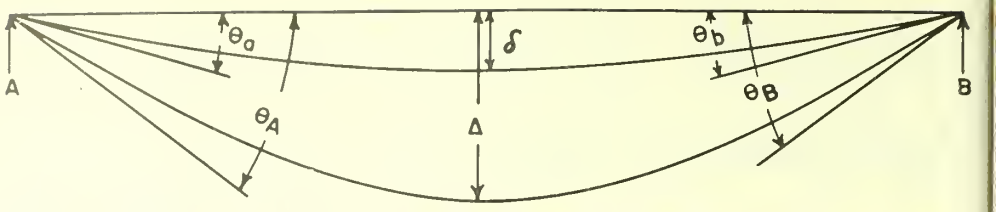

Figure 1.—Relation of rotations of a concrete beam at the end supports to deflections at the center of the span.

Development of Theory

Nonelastic deflection of a concrete beam is produced by nonelastic fiber deformations within the beam just as elastic deflection is produced by elastic fiber deformations. Nonelastic fiber deformations in concrete may be produced by three separate actions: shrinkage due to the concrete setting and drying, creep due to dead load stresses, and plastic flow in areas of very high stresses (*1*).[1] The shrinkage effect is much greater initially than the effects of creep and plastic flow. The drying of concrete is more complete in thin sections; therefore, more shrinkage occurs in a thin section such as a floor slab than in the thicker sections of a beam. The shrinkage effect on the deflection of a span probably starts when the concrete is only a few hours old or near the time it enters the final set period.

Shrinkage, creep, and plastic flow

Shrinkage in thin sections of concrete develops at a fairly rapid rate for the first few months and continues at a decreasing rate for an indefinite period. Most of the shrinkage develops in the first 8 or 9 months.

Creep, when separated from shrinkage, is a delayed deformation similar to elastic deformation and is produced by the same stresses causing elastic deformation. It is a continuing action over an indefinite period. At working stresses, creep is proportional to stress. The effect of creep may be taken into account by reducing the value of the modulus of elasticity of the concrete. The stress in the reinforcing steel is increased through an increase in the value of n, the ratio of the modulus of elasticity of the steel to the modulus of elasticity of the concrete.

Plastic flow in concrete is that deformation which occurs at very high stress (above 70 percent of ultimate), the concrete acting as a viscous solid. Plastic flow deformations are not proportional to stress. If plastic flow occurs in continuous concrete spans, it will probably be at points of high concrete stress produced by the combined dead load, live load, impact, and shrinkage negative moments. Plastic flow at these points will produce a loss in continuity in the spans, which in turn causes the dead load deflections to increase.

In thin concrete members, such as floor slabs, the shrinkage factor due to the concrete setting and drying may be as high as 0.0006 (*1, 2, 3*). In the lightly reinforced top flange portions of a beam there is little or no opposition to concrete shrinkage. Shrinkage in this flange will temporarily relieve the concrete of carrying a portion of its compression load, causing the reinforcing steel to take more compression until it is stressed beyond its elastic limit; the steel is then deformed and the concrete again acts to carry its proportion of the compression load. Thus, the shortening of the top flange lengths is equal to the shrinkage in the concrete. The bottom flange of a beam is heavily reinforced in the positive moment areas. Shrinkage will increase the tension stresses in the concrete and tend to cause cracking. Creep will tend to nullify shrinkage in the bottom flange. Shrinkage will have little or no effect on the deformations within the reinforcing steel or the length of a given portion of the bottom flange of the beam.

Rotation of beams at end supports

A simple span beam cannot deflect from any cause without producing rotation at the supports. The amount of rotation at the supports, as well as the deflection at any point along the span, is the summation of the effect on rotation or deflection of each of the individual changes in fiber lengths, regardless of what produces these changes. Since shrinkage may be assumed to occur uniformly along the positive moment lengths of a beam, the shape of the final deflection curve will be more nearly circular than parabolic as in elastic deflection (*4*). Therefore, the ratios of the final rotations θ_A and θ_B (fig. 1) at the beam ends to the elastic ratios θ_a and θ_b, respectively, will be greater than the ratio of the final deflection, Δ, to the elastic deflection, δ, at the center of the span. If Δ equals 4δ, $\theta_A > 4\theta_a$ and $\theta_B > 4\theta_b$.

In the simple span the rotations, θ_A and θ_B, have no effect on the stresses within the beam. However, in the indeterminate structure (fixed-end spans and continuous spans) the effect of this additional rotation on the stresses of the beam may be tremendous. The bending moment pattern is governed by the end rotations of the spans, considered as simple spans. In the conjugate beam method of stress analysis, the fixed-end moments of a beam can be determined directly from the end rotations. In the fixed-end moment formula $M^F_{AB} = \theta_a K_a - C_{BA}\theta_b K_b$, it can be seen that increasing the end rotations will increase the fixed-end moments proportionally. K_a and K_b, the Cross stiffness factors, and C_{BA}, the Cross carryover factor, are fixed by the moment-of-inertia pattern of the beam which does not change.

[1] Italic numbers in parentheses refer to the list of references on p. 196.

In the continuous span, the fixed-end moment distribution factors are determined from the Cross carryover and stiffness factors of the beam. These factors are not affected by fiber deformation. Therefore, the negative moments of a continuous span are in proportion to the fixed-end moments, and nonelastic deformations from shrinkage in the positive moment lengths of a beam will increase the negative moments. When combined dead load, live load, impact, and shrinkage moments are sufficient to stress the negative moment reinforcing steel beyond the elastic limit, the reinforcing steel will undergo nonelastic elongation and the concrete will crack.

Nonelastic elongation of steel

A very slight nonelastic elongation of the reinforcing steel has two immediate effects. It causes a portion of the continuity of the spans to be lost which increases the end reactions and, at the same time, causes a shifting of the neutral axis of the beam away from the reinforcing steel, thus increasing the effective depth, jd, and reducing the stress in the reinforcing steel. Restoration of a portion of the reactions lost through the initial effect of shrinkage reduces the moments produced by shrinkage. The stress in the concrete over the interior piers is increased by the increase in negative moments through the effect of shrinkage. The shifting of the neutral axis of the beam produced by yielding of the reinforcing steel further increases the concrete stress. Plastic flow which may occur from the high stresses now existing in the concrete over the interior piers will cause a further loss of continuity and an increase in the end reactions.

In some spans the loss of end reaction through the initial effect of shrinkage may be more than completely restored. The spans will then act partially as simple spans. These actions may temporarily stabilize under dead load with the reinforcing steel stresses at the elastic limit of the steel. Then as live loads come onto the spans, the negative moments produced by the live loads will cause yielding in the steel. Since yielding in the reinforcing steel has the immediate effect of reducing the stress producing the yield, it can be seen that the nonelastic elongation of the steel will be small. The modulus of elasticity of the steel probably increases as the steel yields, thus further limiting the size of the cracks in the concrete.

This chain action started by shrinkage in the concrete will stabilize at some point depending on the design. In designs with sufficient negative moment reinforcing steel to hold the steel stress below the elastic limit and the concrete stress below the plastic limit, the action ends with the shrinkage in the concrete and with only a small change in deflections through the actions of creep and shrinkage. In most bridge spans designed to maximum allowable stresses by the elastic theory of stress analysis, cracking of the slabs in the negative moment areas will first develop, thus permitting a small increase in deflections. These cracks may appear as early as 3 months after pouring. Then plastic flow may occur in the concrete at points of high negative moments produced by shrinkage and through an increase in concrete stress caused by the shifting of the neutral axis. The loss of continuity through these actions increases the positive moments and thus increases the dead load deflections.

In spans designed for dead load stresses only, shrinkage may increase the negative moments sufficiently to move the points of inflection beyond the cutoff points of the negative moment reinforcing steel and cause collapse of the spans. The nonreinforced concrete may crack due to its inability to carry the tension stress imposed. The cracking of the beams reduces the areas available to resist shear. The second action leading to collapse of the spans is shear failure through the reduced areas (*5*).

Application of Theory

The increase in negative moments from nonelastic deformations results from a reduction in the dead load end reactions (*6*). This reduction in the end reactions can be determined by considering the beam as weightless and applying a shrinkage factor of e to the lightly reinforced top flange portions of the beam. Let c equal the depth of the beam. Thus, $c/c=M$. Then apply M as a uniform load over the lightly reinforced compression flange lengths of a conjugate beam (*7*), as shown in figure 2(a).

Three-span bridge system

Deflection Y_2 at the ends of the spans equals the moment under loads M of the conjugate beam at R_2 (fig. 2(b)) in a symmetrical system of spans. The curve represents the new physical shape of the unloaded system of spans after shrinkage has occurred, and Y_2 is the distance the ends of the girders would rise off the bearings at R_1 and R_4 if the girders were weightless. The effect on dead load moments within the system caused by this change in physical shape is the same as would occur if piers R_1 and R_4 each settled an amount equal to Y_2.

Forces P are applied at the beam ends to reduce deflection Y_2 to zero, as shown in figure 2(c). P is the reduction in dead load end reaction.

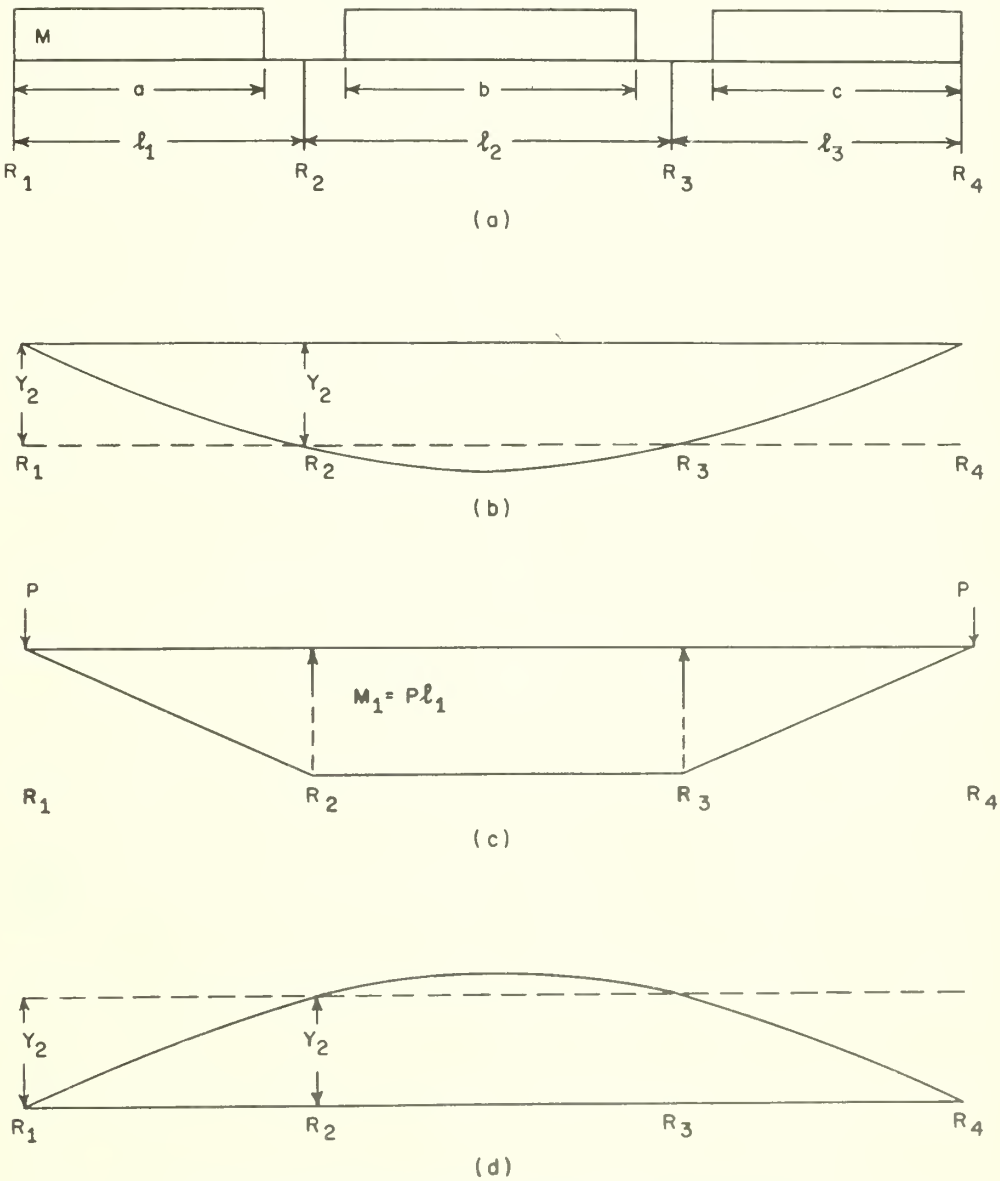

Figure 2.—Theory illustrated for a three-span bridge system.

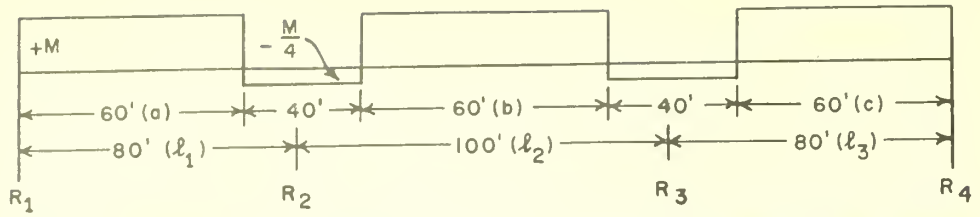

Figure 3.—Illustration of a conjugate beam loaded with positive and negative nonelastic weights.

The deflection Y_2, indicated in figure 2(d), at the ends of the spans produced by loads P equals the moment of the conjugate beam at R_2 under loads M_1 divided by EI. (E=modulus of elasticity of the concrete.) For constant moment of inertia I,

$$Y_2=\frac{2Pl_1^3+3Pl_1^2l_2}{6EI} \qquad \text{Formula 1}$$

When the system of spans is not symmetrical, l_1 does not equal l_3 or the variation in moments of inertia is not symmetrical about the center of the system, shrinkage deflections Y will have to be calculated at both R_1 and R_4. These deflections are then treated the same as similar deflections produced by pier settlements.

As an example, in an 80-100-80-foot continuous I-girder designed for shrinkage forces, the depth of the beam c through the lightly reinforced compression flange lengths was 61.25 inches. Lengths a, b, and c (figs. 2(a) and 3) were 60 feet, e=0.0006, $M=e/c$, I=685,299 in.^{4}, and E=5,000,000 lb./sq. in.

$$Y_2=90M\times80-60M\times50=4{,}200M$$

$$Y_2=\frac{4{,}200\times0.0006\times144}{61.25}=5.92 \text{ inches}$$

From formula 1:

$$5.92=\frac{2P(80)^3\ 1{,}728+3P(80)^2\ 100\times1{,}728}{6\times685{,}299\times5{,}000{,}000}$$

P=23,924 pounds.

The negative moment due to P equaled 1,914 ft. kips. The dead load negative moment equaled 1,582 ft. kips; hence, the negative moment due to shrinkage was 121 percent of the dead load moment. The allowable stress in the negative moment reinforcing steel for combined dead load, live load, impact, and shrinkage moments was 20,000 p.s.i.

If more accuracy is desired in making these calculations, the shrinkage in the compression flange of the negative moment lengths and the variation in the moment of inertia of the beam may be considered. The following deflection formula which takes into account the variation in moment of inertia in a three-span symmetrical system may be used in calculating the value of P.

$$Y_2=\frac{(\theta_{bb}+\theta_{bb2}+\theta_{cb})Pl_1^2}{EI_0} \qquad \text{Formula 2}$$

In this formula θ_{bb} is the rotation at R_2 of span 1 with a one foot-pound moment at R_2; θ_{bb2} is the rotation at R_2 of span 2 with a one foot-pound moment at R_2; and θ_{cb} is the rotation at R_2 of span 2 with a one foot-pound moment at R_3, with each span considered as being a simple span and with the EI values reduced in the ratio of EI_o=1. I_o is the smallest moment of inertia in the system. The values within the parenthesis are obtained from their respective moment diagrams after the moment ordinates have been reduced by dividing them by the appropriate value of I/I_o. The depth of the beam over interior piers was 81.25 inches. The haunches were on parabolic curves and were 20 feet long. For simplicity in calculating the warpage of the beam due to shrinkage, the depth of the beam over these lengths will be considered as being constant. The compression flange was 24 inches wide and contained 10.92 square inches of reinforcing steel. In view of the thickness of the flange, its less exposed position, and the amount of reinforcing steel, the shrinkage factor e is probably about 0.0002 (2). The negative M ($M=e/c$) is about one-fourth of the M in the positive moment lengths.

The lengths a and c, figures 2(a) and 3, are approximately the lengths between the ends of the spans and the points of dead load contraflexure in spans 1 and 3. The length b is approximately the length between these points in span 2. The locations of the dead load points of contraflexure obtained by the elastic theory will move when the shrinkage effect is added to the elastic behavior of a beam.

$$Y_2=80M\times80-60M\times50+20\times\frac{M}{4}\times10=3{,}450\ M$$

$$Y_2=\frac{3{,}450\times0.0006\times144}{61.25}=4.87 \text{ inches}$$

From formula 2:

$$4.87=\frac{(21.209+27.75+15.95)P\times(80)^2\times1{,}728}{5{,}000{,}000\times685{,}299}$$

P=23,246 pounds; the negative moment due to P is 23,246×80=1,859,680 foot pounds.

The shrinkage factor e on the compression side of the negative moment lengths will be smaller than at the positive moment lengths because of the thicker concrete sections and less exposure to drying. These sections usually have compression reinforcing steel which by the elastic theory carries low stress. Shrinkage in these sections will have the initial effect of forcing the steel to carry more of the compression load. Apparently the steel is not stressed beyond the elastic limit in the early stages of the action. It may be stressed to this point if plastic flow occurs. When the compression reinforcing steel with the added stresses caused by shrinkage in the concrete is stressed below the elastic limit, shortening of the flange will result from the elastic shortening in the reinforcing steel through the action of these added stresses. The factor e will change as the bending moments produced by dead loads and shrinkage effect changes in the lengths; it will have a very small value at points with zero bending moment and will approach the value of the shrinkage factor of the concrete at points with high bending moments.

The beams are usually about twice as deep over the interior supports as they are in the positive moment lengths. A unit shortening of concrete fibers at these points would have only about half the effect of deforming the physical shape of the beam as would the same shortening at the thinner sections. Shrinkage in these sections will reduce the value of Y_2 (fig. 2(b)). However, with the value of negative M ($M=e/c$) being only about one-fourth of the value of M at lengths a, b, and c in figure 2(a), its effect on the value of Y_2 is small. The assumed points of inflection, which are the dividing points between positive and negative nonelastic loads, used in making these calculations must agree with the actual points of inflection of the beam under the combined effect of dead loads and shrinkage. It is usually necessary to make three or four trial calculations, adjusting these assumed points of inflection, until this agreement is reached.

In view of the difficulty at this time of estimating the ultimate value of the shrinkage factor e, which is affected by many factors including the cement and aggregate used, the proportions of the mix (especially the amount of water used in the mix), the curing of the concrete, and the climate, negative M is usually not included in the calculations. The additional cost of including the shrinkage effect over lengths a, b, and c (fig. 2(a)) in the design is usually small, and this procedure is on the side of safety.

Four-span bridge system

The procedure used in calculating the reduction in dead load end reactions produced by shrinkage in a four-span symmetrical system is similar to that used in the three-span system.

In figure 4(a), nonelastic loads M are applied to the span system considered to be a conjugate beam supported at R_1 and R_5. Deflections Y_1 and Y_2 (fig. 4(b)) equal the moments in the conjugate beam at R_3 and R_2, respectively, produced by the nonelastic loads M shown in figure 4(a). Deflection Y_3 equals Y_1 minus Y_2. Deflection Y_1 represents the amount the beam, if weightless, would rise off the bearings at R_1 and R_5. Y_3 is the amount the beam would rise off the bearings at R_2 and R_4. These deflections may be treated as settlements in piers R_1, R_2, R_4, and R_5.

Forces P_1 (fig. 4(c)) are applied at R_1 and R_5 to reduce deflection Y_3 to zero. Y_3 equals

Y_5 minus Y_4 (fig. 4(d)). For constant moment of inertia,

$$Y_3=\frac{P}{6EI}(3l_1l_2^2+2l_2^3) \quad \text{Formula 3}$$

Deflection Y_6 (fig. 4(e)) is the deflection remaining at R_1 and R_5 after loads P_1 have been applied. Y_6 equals Y_1 minus Y_5. It is necessary to apply a second load P_2 at each end to reduce Y_6 to zero. The value of P_2 is found by loading the conjugate beam as illustrated in figure 4(f) and equating the moment at R_3 divided by EI to Y_6. The reduction in dead load reactions at R_1 and R_5 equals P_1+P_2. The increase in negative moment at R_2 equals $-(P_1+P_2)l_1$. The increase in negative moment at R_3 equals $-P_1(l_1+l_2)+C_{BC}P_2l_1$. C_{BC} is the Cross carryover factor from end B to end C of span 2.

General Considerations

The shrinkage effect does not become appreciable until after the spans are several weeks old. Therefore, to prevent overstress in the positive moment areas of the span during the early life, these areas should be designed for dead load, live load, and impact moments only. The concrete stress for combined moments including the shrinkage effect should not exceed allowable limits. Since shrinkage on drying is governed mainly by the unit water content of the mix, the dryest mix feasible should be used in pouring continuous concrete spans.

The tensile stress at the roadway surface will exceed the ultimate tensile strength of the concrete throughout most of the negative moment lengths. The elastic elongation of the negative moment reinforcing steel over these lengths will be distributed in a few large cracks in areas where the concrete is not reinforced. To prevent such large cracks from forming, a portion of the negative moment reinforcing steel should be distributed across the floor slabs between girders. The reinforcing steel should be placed in the upper portion of the floor slabs and as near to the surface of the floor as the main slab reinforcing and the specifications will permit; otherwise objectionable cracks may develop from the normal elastic elongation of the reinforcing steel.

Deflection curves and camber diagrams calculated by the theory of combined elastic and nonelastic deformations are affected very little by the additional effect of shrinkage. The reduction in dead load end reactions as a result of shrinkage produces deflections opposite in direction and approximately equal to those produced directly by shrinkage in the spans.

Pouring Sequence of Concrete

The order of concrete pouring for a three-span system was designed to reduce the hazard of loss through a washout of falsework. The construction joints are placed approximately at dead load points of inflection. The pours are made in the alphabetical sequence indicated in figure 5. Sections A plus B and C plus D are designed to be self-supporting after sections B and D are a few days old. This procedure reduces the hazard of loss to a maximum of two sections and reduces the vulnerable periods to a total of 18 to 20 days. Pours A plus B made in warm

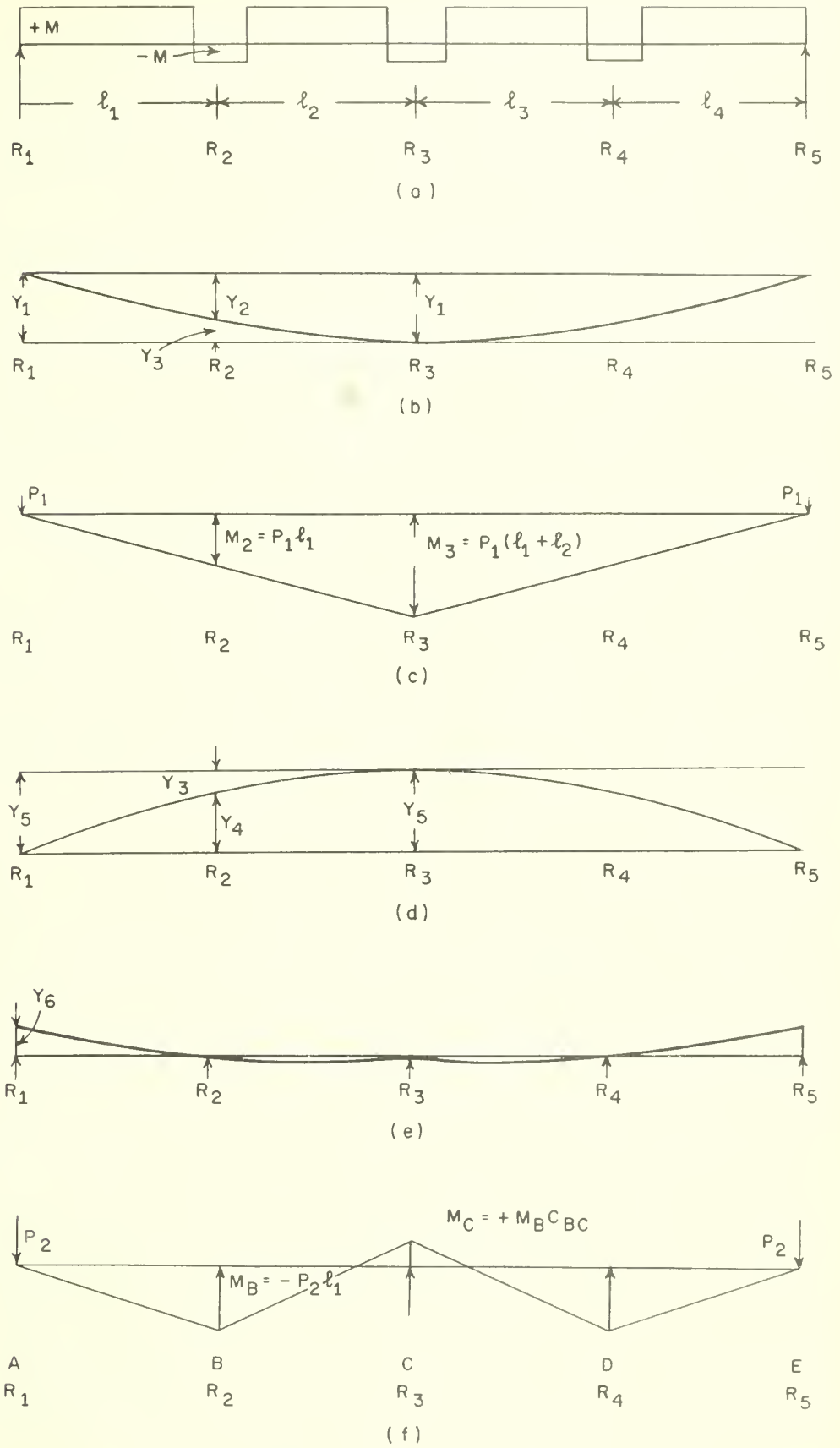

Figure 4.—Theory illustrated for a four-span bridge system.

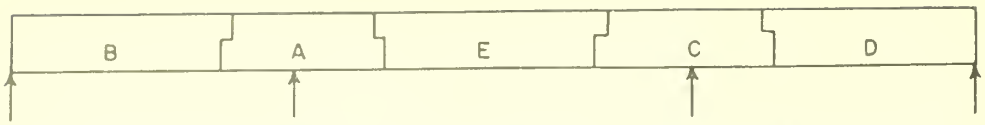

Figure 5.—Sequence of pouring concrete for a three-span bridge system.

weather would probably stand without damage after pour B was about 4 days old.

This pouring sequence allows initial shrinkage in the concrete in sections A, B, C, and D to develop before the spans are connected. However, it is believed that shrinkage in sections B, D, and E, which develops while the concrete is supported by the forms, either produces microscopic cracks in the concrete or places the concrete in tension. These microscopic cracks or tension act to change the physical shape of the beam when the supports are removed and have the same effect on stresses as shrinkage which develops after the spans are completed.

REFERENCES

(1) *Concrete Manual.* U.S. Department of the Interior, Bureau of Reclamation, sixth ed., 1955, ch. 1, par. 8, 11, and 14.

Figure 7, page 17, and figure 17, page 35, of the manual show a drying shrinkage factor of about 0.0006 for a mix containing 325 pounds of water per cubic yard of concrete. This is approximately the mix used in the structures discussed in the present study.

(2) *Concrete for Prestressing,* by Robert F. Blanks, Chief, Research and Geology Division, Bureau of Reclamation. Proceedings of the First United States Conference on Prestressed Concrete, Massachusetts Institute of Technology, Cambridge, Mass., Aug. 1951.

Shrinkage in 6-inch members (fig. 3, p. 145) may be expected to reach 0.0006 between 15 days and 9 months of concrete age. Figure 2 (p. 145) indicates 0.00065 shrinkage in 4- by 4- by 30-inch prisms between 3 and 9 months.

(3) *Study of Shrinkage in Concrete Frames,* by Morgan B. Klock and Robert R. Sheridan. Journal of the American Concrete Institute, vol. 28, No. 8, Feb. 1957, pp. 791–796.

The measured vertical shrinkage factor of a concrete building frame over a 16-year period was 0.000985. Horizontal shrinkage in reinforced concrete lengths was 0.00046. Figure 1 (p. 793) shows that the drying shrinkage factor in the building slabs was 0.00075 at a concrete age of 4 years or about 0.0005 at the end of 1 year.

(4) *Warping of Reinforced Concrete Due to Shrinkage,* by Alfred L. Miller. Journal of the American Concrete Institute, vol. 29, No. 11, May 1958, pp. 939–950.

(5) *Rigid-Frame Failure Raises Questions as to Standard Design Assumptions,* by Edward Cohen. Civil Engineering, vol. 26, No. 2, Feb. 1956, pp. 45–50.

Discussion of the collapse of about 4,000 square feet of roof at the Special Air Materiel Command Warehouse at Wilkins Air Force Depot, Shelby, Ohio.

(6) *Plastic Flow (Creep) of Reinforced Concrete Continuous Beams,* by G. W. Washa and P. G. Fluck. Journal of the American Concrete Institute, vol. 27, No. 5, Jan. 1956, pp. 549–561.

The authors indicate in conclusion No. 3 (p. 561) that the mid-span reaction of the two-span continuous beams increases about 5 percent. These beams carried a superimposed uniform load in addition to the dead load of the beams. This load should be considered as a live load and its effect on the reactions should be removed when changes in dead load reactions are being considered. When the effect of superimposed load on the type Y beams is removed, the increase in mid-span dead load reaction is about 12.4 percent. The decrease in dead load end reactions is 20.5 percent and the increase in the dead load negative moment over the mid-span support is 61.5 percent.

In one of the type X beams the increase in dead load negative moment is 52 percent. A shrinkage factor of 0.0006 applied over the positive moment lengths and 0.0003 over the negative moment lengths of these beams gives approximately these same increases in dead load negative moments. Figure 5 (p. 556) shows that most of the increase in mid-span reaction developed during the first 200 days. The curve showing this increase in reaction with time is similar to curves showing increase in shrinkage with time.

(7) *Strength of Materials, Part I, Elementary Theory and Problems,* by S. Timoshenko. D. Van Nostrand Company, Inc., New York, third ed., ch. 5, pp. 137–175.

Reports to Congress

Three reports have been submitted to the Congress by Secretary of Commerce Lewis L. Strauss during 1959. The titles of the reports are the *Progress Report on the Federal-Aid Highway Program,* the *Third Progress Report on the Highway Cost Allocation Study,* and *The Federal Role in Highway Safety.*

The *Progress Report on the Federal-Aid Highway Program,* prepared by the Bureau of Public Roads pursuant to the requirement of section 116 of the Federal-Aid Highway Act of 1956, describes the progress of the Federal-aid highway program in the 2½ years since passage of that act. Of particular interest is information on the status of the National System of Interstate and Defense Highways, presented in tabular form and on two large maps printed in color.

This report, printed as House Document No. 74, is for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington 25, D.C., at 70 cents a copy.

The *Third Progress Report of the Highway Cost Allocation Study* was undertaken by the Bureau of Public Roads pursuant to the requirement of section 210 of the Highway Revenue Act of 1956.

The purpose of the highway cost allocation study is to make available to the Congress information on the basis of which it may determine what taxes should be imposed by the United States, and in what amounts, in order to insure, insofar as practicable, an equitable distribution of the tax burden among various classes of persons using the Federal-aid highways or otherwise deriving benefits from them. The final report of the study is to be completed in January 1961.

The 66-page third progress report presents in summary form the information obtained from extensive studies of the numbers, types, and fuel consumption of motor vehicles registered, and the types, volumes, and weights of motor-vehicle traffic on the several highway systems. Forecasts of vehicles and travel are included. The report also presents a digest of a series of studies of the economic impact of highway improvement.

This report, printed as House Document No. 91, is available from the Superintendent of Documents at 35 cents a copy. Also available from the same source is the 131-page First Progress Report, dated February 28, 1957, which presents a wealth of background material and a statement of the problem and plans for the study (35 cents per copy). The 6-page Second Progress Report, dated February 28, 1958, reported very briefly the nature of work underway.

The Federal Role in Highway Safety has been printed as House Document No. 93 and is for sale by the Superintendent of Documents at 60 cents a copy. This study, undertaken by the Bureau of Public Roads pursuant to section 117 of the Federal-Aid Highway Act of 1956, was made to determine what actions could be taken by the Federal Government to promote the public welfare by increasing highway safety.

The study evaluates the dimensions of the traffic accident problem and the relation of the driver, vehicle, and highway in connection thereto. It also evaluates the current safety activities and presents an adequate safety program. The findings give support to judgments formed over the past years on many highway safety matters, and also exposes widely held misconceptions of traffic accidents.

U. S. GOVERNMENT PRINTING OFFICE: 1959

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1952, 25 cents. 1955, 25 cents. 1958, 30 cents.

PUBLICATIONS

A Report of Factors for Use in Apportioning Funds for the National System of Interstate and Defense Highways, House Document No. 300 (1958). 15 cents.

Bibliography of Highway Planning Reports (1950). 30 cents.

Braking Performance of Motor Vehicles (1954). Out of print.

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VOL. 30, NO. 9 AUGUST 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

Recently completed East Bridge of the MacArthur Causeway which joins Miami Beach and Miami, Fla. (Miami Beach in the foreground)

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Published Bimonthly

Vol. 30, No. 9 *August 1959*

C. M. Billingsley, *Editor*

IN THIS ISSUE

Properties of Highway Asphalts—Part I. 85–100 Penetration Grade 197

Maximum Wind Speeds to Consider in Designing Highway Signs (*see text on page 212*) . . . 208

North Dakota's Use of Aerial Inventory for County General Highway Maps 210

New Publication: Highway Statistics, 1957 . . 212

Characteristics of Traffic Entering and Leaving the Central Business District 213

U.S. DEPARTMENT OF COMMERCE
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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

Properties of Highway Asphalts—Part I, 85-100 Penetration Grade

BY THE DIVISION OF PHYSICAL RESEARCH
BUREAU OF PUBLIC ROADS

Reported[1] by J. YORK WELBORN, Chief, Bituminous and Chemical Branch, and WOODROW J. HALSTEAD, Head, Bituminous Materials and Chemical Section

This article is the first of a series of a general study of asphalts produced for highway purposes. A total of 323 samples from 105 refineries were collected by the States for test purposes. Of this number, 146 samples were of the 85–100 penetration grade. This article includes the commonly determined test characteristics of the materials of this penetration grade as well as the results of some of the better known special tests.

The data presented are believed to be valuable as an indication of the range of test characteristics of asphalts that might normally be expected in all regions of the country, as an indication of the usefulness of various specification requirements, and also as a guide for further research directed toward the establishment of "quality" tests for asphalts.

A discussion of the suitability of some of the special quality requirements that have been suggested and used in some specifications is included. No overall conclusions have been drawn since the need for more complete data is apparent.

THIS REPORT catalogs the properties of asphalt cements of the 85–100 penetration grade produced in the United States for use in highway construction. The information presented here is the first report of a comprehensive study of asphalt undertaken by the Division of Physical Research in 1954. Reports on other phases of this study will be published at a later date.

The results of a similar study conducted by the Division of Physical Research in the late thirties were published in 1940 and 1941, and followed by another report published in 1946 (*1–3*).[2] These studies proved useful in that they showed the conformity of the asphalts to specification requirements and presented the general range in test characteristics. Values for the standard tests and also for a large number of special tests either in use or proposed at that time were included.

The present survey provides similar up-to-date information on a national scope, including the properties of asphalts produced from various crude sources and methods of refining in current use in the United States.

Sources of Samples

The samples of asphalt cement for the current study were obtained for the Division of Physical Research by the regional offices of the Bureau of Public Roads. The States within each region cooperated to the fullest extent by collecting representative samples from the producers supplying their material. Although it is likely that some producers are not represented, the materials received are a relatively complete sample of asphalt cements produced and used during 1954 and 1955. New producers, crude sources, and changes in refinery techniques instituted since that time naturally are not reflected in the properties of these materials. However, it is believed that the materials received are essentially the same as those in use today.

Some of the samples of asphalt were obtained by State highway departments from actual construction projects, and others were obtained at the refineries. At the time of sampling, information concerning the source of the crude petroleum and the general refinery method used to manufacture the asphalt was requested from the producers and received from most of them.

Initially the Bureau's regional offices were requested to collect samples of only the 60–70, 85–100, and 120–150 penetration grades. When it was found that other penetration grades were used extensively by some States, the selection of grades was left to the discretion of each Regional Engineer.

A total of 323 samples from 105 refineries were received. The penetration grades and the number of samples of each grade were as follows: 60–70 grade, 59 samples; 70–85 grade, 33; 85–100 grade, 146; 100–120 grade, 7; 120–150 grade, 62; and 150–200 grade, 16.

Nearly all producers supplying asphalt for this study included one or more samples in the 85–100 penetration range. Since this group of samples is considered the most representative of all asphalt cements now produced and used in the United States, it was selected for the initial study. It is planned that subsequent reports will include the properties of the other grades as well as more intensive study of selected types or particular characteristics of asphalts.

The data in this report include only those test characteristics in general use as specification requirements or those that are being used by some agencies in an effort to obtain better materials. All testing was performed according to ASTM or AASHO standard methods of test.

The results of the tests on the 146 samples of 85–100 penetration asphalt are tabulated in tables 1 and 2. The data in the tables are grouped according to regions.

There was some duplication of samples within a region because some of the States collected material from a particular refinery using the same crude source and refining method. When this occurred only one sample is included in table 1 and the others are included in table 2. However, in some cases where a particular refinery supplied an asphalt to more than one region, as represented by different samples, data for that asphalt are included in each region. In most cases the test results indicated that the materials were essentially the same, but in a few cases there were significant differences. The variations in test characteristics generally occurred in samples from those refineries which reported the use of more than one type of crude or blends of crudes. Since samples from different regions were taken at different times, it is likely that there were actual differences in the crudes. This would account for asphalts having different characteristics.

Tables 1 and 2 show the refinery by a code number, the source of the crude or crudes used, and the general method of producing the asphalt. Although in many cases more specific information was given as to the crude source and refinery processes, only the basic information that generally characterizes the material is included. It is recognized that differences can and do exist within the range of the geographical location of the crude sources indicated, and also within the meaning of the general terms used to describe the method of refining.

Of the 119 samples included in table 1, 73 were reported to be refined by vacuum and/or steam distillation, 15 by vacuum and/or steam distillation with some blowing, 3 by vacuum and/or steam with fluxing, and

[1] This article was presented before the Association of Asphalt Paving Technologists, Denver, Colo., January 1959.

[2] Italic numbers in parentheses refer to the list of references on p. 207.

511782—59——1

Table 1.—Test characteristics of 85–100 penetration grade asphalts

B.P.R. region [1]	Sample identification [2]	Source of crude	Method of refining [3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl_4	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
																Penetration of residue			Tests on residue			
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup		Loss	Value	Percent of original	Loss [4]	Softening point	Ductility, 5 cm. per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	*Cm.*	° *F.*	*Sec.*		° *F.*	° *F.*	*Pct.*	*Pct.*			*Pct.*	° *F.*	*Cm.*		
Region 1	1 (2)	Venezuela	V, S	87	30	34	195	14.5	120	239	1.031	450	530	99.86	0.07	76	87	0.23	131	194	52	60
	2 (4)	do	V, S	88	32	36	225	99.5	118	137	1.018	370	455	99.93	.58	66	75	2.04	131	250+	33	38
	3 (22)	Mexico	S	87	39	45	248	51.5	121	318	1.037	445	500	99.92	.12	77	89	.55	134	138	52	60
	4 (22)	Venezuela	S	87	37	43	158	71.0	120	250	1.034	435	485	99.98	.16	76	87	.75	132	200	48	55
	5 (22)	Venezuela, Mexico	S	95	34	36	212	36.0	117	247	1.034	435	495	99.84	.12	81	85	.59	131	212	53	56
	6 (30)	Colombia		86	30	35	180	79.0	115	200	1.005	525	580	99.75	.07	81	94	+.10	122	250+	61	71
	7 (32)	Venezuela	V	91	31	34	233	62.5	115	178	1.024	510	550	99.90	.02	83	91	.10	124	243	58	64
	8 (77)			91	37	41	217	18.5	117	213	1.020	425	545	99.87	.18	78	86	.50	127	188	53	58
	9 (77)	Venezuela, Texas	V, S, O	93	35	38	150	22.3	116	198	1.022	415	540	99.92	.20	80	86	.50	130	226	53	57
	10 (86)	Coastal	V	91	35	38	204	16.0	117	214	1.017	520	595	99.91	.00	83	91	.02	123	250+	61	67
	11 (90)	Venezuela	V	85	35	41	250+	14.5	118	202	1.016	475	555	99.92	.07	75	88	.14	130	208	54	64
	12 (110)		S	85	33	39	177	27.0	117	202	1.020	475	575	99.89	.03	75	88	.01	125	250+	56	66
Region 2	13 (1)	Venezuela		89	32	36	175	18.3	120	255	1.033	480	530	99.85	.03	79	89	.18	132	175	55	62
	14 (3)	do	V, S	92	32	35	185	29.5	117	232	1.030	440	490	99.85	.13	82	89	.70	130	182	52	57
	15 (9)	do	S	96	34	35	189	33.5	118	249	1.036	420	490	99.88	.13	80	83	.64	132	176	53	55
	16 (13)	Illinois	V, S	83	30	36	187	6.5	118	169	1.005	495	625	99.82	.06	71	86	.01	129	154	53	64
	17 (14)			93	34	37	196	63.0	116	166	1.022	535	565	99.95	.01	81	87	+.04	123	205	61	66
	18 (17)			87	40	46	116	7.3	124	239	.992	425	630	99.89	.07	75	86	.08	135	29	57	66
	19 (31)	Venezuela	V	94	37	39	205	15.5	116	209	1.025	510	570	99.89	.00	82	87	.03	126	148	58	62
	20 (70)			87	31	36	199	47.0	117	204	1.019	510	590	99.96	.01	78	90	.01	125	229	59	68
	21 (81)			91	40	44	197	7.0	117	207	1.009	540	620	99.78	.00	84	92	+.07	130	125	61	67
	22 (82)			95	35	37	154	9.8	119	191	.994	545	630	99.96	.00	85	89	+.05	124	169	66	69
	23 (87)	Gulf Coast	V	81	37	46	190	11.3	119	198	1.016	530	585	99.87	.03	75	93	.04	127	208	57	70
Region 3	24 (6)	Venezuela, Mississippi	V, S, B	94	36	38	200	32.5	120	251	1.032	460	500	99.87	.07	87	93	.36	131	156	57	61
	25 (10)	Venezuela, Mexico	S	95	40	42	171	35.5	120	298	1.036	440	505	99.96	.12	80	84	.56	134	160	54	57
	26 (11)	Texas	S	87	37	43	231	13.3	118	219	1.028	375	590	99.93	.15	73	84	.19	129	250+	51	59
	27 (33)	Venezuela	V, S	93	34	37	195	88.0	113	153	1.025	495	545	99.88	.02	82	88	.13	123	221	61	66
	28 (33)		S	89	31	35	184	152.0	116	172	1.023	385	540	99.90	.10	77	87	.21	125	250+	53	60
	29 (42)	Mississippi, Alabama	V, S	89	37	42	177	9.8	119	259	1.036	445	550	99.92	.07	78	88	.16	129	183	55	62
	30 (63)	Mississippi	S	91	46	51	222	9.3	123	265	1.029	440	505	99.90	.11	79	87	.45	137	70	53	58
	31 (65)			93	46	49	155	11.0	119	247	1.019	450	520	99.65	.08	78	84	.33	136	77	52	56
	32 (69)	Venezuela		94	36	38	250+	76.5	117	200	1.018	500	580	99.97	.03	85	90	.04	125	250+	57	61
	33 (83)	Mississippi	V, S	96	42	44	180	13.5	119	244	1.033	470	530	99.95	.03	88	92	.12	131	117	59	61
	34 (102)	do	S	87	45	52	140	8.3	124	268	1.031	390	490	99.96	.20	74	85	.73	140	40	47	54
	35 (103)	do	S	92	46	50	197	18.5	122	262	1.029	420	495	99.72	.18	76	83	.59	137	71	50	54
Region 4	36 (1)	Venezuela	V, S	87	34	39	218	19.3	120	264	1.034	450	525	99.92	.03	75	86	.14	130	184	52	60
	37 (7)	Texas	V	80	37	46	136	7.3	125	195	1.029	490	575	99.88	.02	73	91	.00	136	68	53	66
	38 (8)	Midcontinent	V, O	90	32	36	170	6.3	120	189	1.000	510	605	99.76	.02	79	88	.00	129	120	56	62
	39 (13)			88	27	31	193	13.3	116	188	1.001	460	675	99.98	.02	77	87	.02	124	234	59	67
	40 (15)	Wyoming	V	83	27	33	185	10.8	118	169	1.027	435	585	99.89	.05	72	87	.11	126	250+	50	60
	41 (31)		V, S	93	36	39	202	25.0	116	192	1.018	510	590	99.83	.00	82	88	+.05	123	250+	59	63
	42 (44)	Canada		82	20	23	190	5.0	123	110	.999	570	660	[5] 98.00	.01	67	82	+.08	131	160	50	61
	43 (44)	do		83	18	22	176	5.8	118	108	1.009	545	620	99.62	.04	73	88	.01	126	157	49	59
	44 (45)	Wyoming	V	81	26	32	150	15.8	117	193	1.026	450	630	99.90	.03	71	88	+.06	126	239	49	60
	45 (46)	Arkansas	V, S	90	26	29	212	6.3	115	220	1.020	545	670	99.91	.01	78	87	.17	125	200	60	67
	46 (59)	Texas	S, O	89	36	41	207	9.5	117	222	1.007	470	580	99.95	.07	80	90	.14	124	205	60	67
	47 (59)	Wyoming, Texas	S, O	94	34	36	137	10.0	116	140	1.019	505	565	99.91	.02	83	88	.00	126	124	59	63
	48 (60)	Midcontinent	V, S, O	90	39	43	138	6.8	120	207	.997	415	620	99.74	.09	77	86	.10	134	37	53	59
	49 (64)	Wyoming, Texas	V, O, B	93	26	28	169	6.3	117	133	1.014	535	630	99.89	.00	80	86	.11	126	180	55	59
	50 (67)	North and West Texas	V, O, B	98	29	30	140	8.0	114	135	1.015	435	610	99.88	.06	78	80	.09	125	166	51	52
	51 (70)	Venezuela	V, S, B	89	36	40	224	78.0	118	206	1.017	475	555	99.92	.03	78	88	.06	128	215	56	63
	52 (74)	Wyoming, Oklahoma	V	85	22	26	222	5.0	115	146	1.016	440	630	99.75	.05	70	82	.02	127	250+	45	53
	53 (89)	Midcontinent	V, O	86	34	40	151	7.3	119	195	.995	545	635	99.88	.01	79	92	+.03	126	150	59	69
	54 (93)	Wyoming	V	83	29	35	182	8.5	117	162	1.027	550	595	99.92	.04	71	86	.03	128	250+	48	58
Region 5	55 (7)			86	31	36	197	10.3	112	244	1.027	510	605	99.97	.02	76	88	.00	127	198	55	64
	56 (12)	Kansas	V	86	33	38	166	4.8	121	180	1.004	525	665	99.70	.01	75	87	+.03	139	13	53	62
	57 (25)	do	V	94	38	40	220	7.5	118	200	1.001	495	650	99.93	.09	79	84	.10	132	40	58	62
	58 (36)	do	V, B	87	35	40	153	6.0	119	205	1.005	500	610	99.81	.02	77	89	.00	133	55	54	62
	59 (39)	Wyoming	V	98	32	33	160	27.0	112	133	1.034	435	530	99.90	.15	79	81	.37	126	211	48	49
	60 (46)			92	28	32	210	8.5	115	216	1.029	610	680	99.94	.00	76	83	+.09	124	200	59	64
	61 (47)	Canada	V	88	19	22	151	5.3	116	99	1.012	585	625	99.89	.01	71	81	+.12	126	184	50	57
	62 (55)	Kansas, Oklahoma, Texas	V, P, B	85	28	33	161	6.8	118	211	1.004	480	660	99.88	.01	74	87	.16	126	184	56	66
	63 (61)			84	31	37	149	9.8	118	171	1.013	460	580	99.88	.05	74	88	.10	130	153	52	62
	64 (76)	Kansas	V, S	97	37	38	155	8.0	117	201	1.004	420	630	99.86	.04	80	82	.00	130	76	58	60
	65 (78)	do	V, S, B	90	38	42	153	5.5	121	229	.998	465	615	99.80	.02	79	88	.02	130	82	60	67
	66 (107)	do	V	90	34	38	129	6.5	120	231	.998	485	635	99.95	.02	77	86	.00	130	80	58	64

Table 1.—Test characteristics of 85–100 penetration grade asphalts—Continued

B.P.R. region [1]	Sample identification [2]		Source of crude	Method of refining [3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl_4	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
																Loss [4]	Penetration of residue		Loss [4]	Tests on residue			
					100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup			Value	Percent of original		Softening point	Ductility, 5 cm. per min., at 77° F.	Penetration at 77° F.	Percent of original penetration
								Cm.	*Cm.*	° *F.*	*Sec.*		° *F.*	° *F.*	*Pct.*	*Pct.*			*Pct.*	° *F.*	*Cm.*		
Region 6	67	(7)	Texas	V	90	34	38	201	20.8	122	243	1.029	460	575	99.78	0.05	78	87	0.08	129	192	55	61
	68	(21)	do	V, S, O	89	30	34	178	6.3	117	153	1.009	485	585	99.61	.03	77	87	.05	130	70	50	56
	69	(23)	do	V, S	87	20	23	112	12.5	116	120	1.029	535	585	99.67	.01	77	89	+.02	125	178	53	61
	70	(27)	Midcontinent	V, S, O	87	28	32	150	6.8	120	208	1.005	510	635	99.82	.01	75	86	+.03	127	250+	53	61
	71	(28)	do	V, P, B	87	26	30	200	112.0	116	190	1.010	525	630	99.82	.00	79	91	+.08	121	190	61	70
	72	(29)	Oklahoma	V, P, O, F	95	47	49	139	7.5	121	203	1.008	455	520	99.77	.06	82	87	.28	135	35	58	61
	73	(35)	Texas	V, S	93	33	35	234	10.3	116	239	1.025	465	625	99.82	.01	83	89	.05	125	209	60	64
	74	(41)	do	V, P, O	94	22	23	250+	4.3	117	263	1.005	525	640	99.73	.01	81	86	+.03	127	181	64	68
	75	(46)	Arkansas	V, S	88	26	30	215	6.5	117	228	1.018	600	675	99.90	+.01	80	91	+.01	125	219	52	59
	76	(48)	Oklahoma	V, O	93	48	52	107	7.5	125	257	1.002	425	475	99.81	.14	82	88	.70	138	20	57	61
	77	(49)	Arkansas	V, S, B, O	94	35	37	213	9.8	117	238	1.016	550	635	99.93	.00	85	90	.06	124	185	65	69
	78	(54)	Mississippi	S	92	48	52	139	9.3	122	240	1.030	400	490	99.84	.12	78	85	.42	138	36	53	58
	79	(57)	Gulf, Midcontinent	V, P, B, O	90	23	26	145	6.0	118	123	1.001	485	565	99.53	.07	80	89	.20	127	114	63	70
	80	(65)	Mississippi	S	90	47	52	180	17.0	114	246	1.030	445	520	99.83	.07	81	90	.20	128	102	57	63
	81	(72)	Texas, Kansas	V, P, O	91	32	35	120	3.5	119	303	.984	550	655	99.48	+.01	84	92	+.08	128	80	63	69
	82	(85)	Mexico, Gulf	V	91	30	33	160	9.3	115	142	1.027	465	530	99.72	.00	85	93	.05	122	182	60	66
	83	(87)	Gulf Coast	V	93	34	37	184	79.5	115	207	1.027	530	605	99.73	.01	82	88	+.03	124	250+	60	65
	84	(92)	Texas	V, O	99	33	33	162	160+	113	85	1.011	450	495	99.65	.09	88	89	.31	118	232	69	70
	85	(95)	do	V, S, O	90	37	41	132	6.3	118	167	1.000	480	590	99.88	.05	80	89	.03	127	127	58	64
	86	(97)	Oklahoma	V	85	27	32	207	8.8	118	238	1.001	525	680	99.51	.01	73	86	.08	124	192	55	65
	87	(104)	do	V, S	88	34	39	166	12.3	118	180	1.012	515	570	99.86	.00	77	88	+.05	128	162	55	63
	88	(106)	do	V, B	81	32	40	172	5.3	122	213	.995	540	645	99.95	.06	69	85	.07	132	104	48	59
	89	(111)	Arkansas	V	89	30	34	138	11.0	116	236	1.021	560	660	99.91	.00	81	91	+.09	124	160	64	72
Region 7	90	(24)	California	V, S	92	36	39	237	49.0	117	210	1.028	435	480	99.88	.15	83	90	.88	129	120	51	55
	91	(40)	do	V, S, B, O	87	27	31	218	109	111	109	1.017	480	575	99.59	.05	78	90	.05	119	190	56	64
	92	(58)	do	V, S	91	30	33	178	132	113	148	1.020	455	520	99.86	.13	77	85	.42	124	242	50	55
	93	(66)	do	V, S, F	89	27	30	216	160+	111	90	1.014	515	565	99.87	.02	82	92	.02	118	250+	58	65
	94	(84)	do	V, S	91	29	30	245	13.8	118	196	1.032	440	480	99.92	.16	75	82	.76	131	160	46	51
	95	(99)	do	S, F	86	31	35	200	14.8	119	210	1.029	450	495	99.86	.10	77	90	.40	131	192	48	55
Region 8	96	(5)	do	V, S	93	23	25	190	8.8	113	104	1.011	470	520	99.93	.05	82	88	.09	120	250+	56	60
	97	(5)	do	V, S	94	24	26	180	250+	113	114	1.017	395	460	99.82	.29	80	85	1.62	128	250+	43	46
	98	(38)	Wyoming	V	96	31	32	220	10.3	116	140	1.033	505	570	99.92	.05	84	88	.03	125	250+	57	59
	99	(43)	do	V	86	24	28	202	44.5	116	196	1.028	465	555	99.91	.05	76	88	.07	125	159	53	62
	100	(100)	California	S	88	29	33	245	26.0	120	228	1.031	410	450	99.69	.27	74	84	1.03	136	125	43	49
	101	(105)	Montana	V	93	26	28	198	6.3	117	154	1.022	430	590	99.86	.07	82	88	.08	125	170	57	61
Region 9	102	(23)	Texas	V, S	92	25	27	121	12.3	116	112	1.031	480	570	99.85	.05	76	83	.06	124	174	57	62
	103	(24)	California	V, S	91	29	32	176	6.8	120	214	1.029	400	440	99.93	.34	75	82	1.63	139	61	39	43
	104	(37)	Wyoming	V	80	25	31	241	7.0	117	170	1.031	495	585	99.90	.02	69	86	.00	126	250+	46	58
	105	(39)	do	V	93	30	32	214	9.3	115	154	1.034	430	515	99.92	.12	75	81	.30	129	235	47	51
	106	(43)	do	V	92	27	32	206	27.0	115	191	1.029	510	560	99.91	.02	74	80	.00	124	240	55	60
	107	(50)			89	20	23	135	5.5	113	123	1.017	455	625	99.91	.04	78	88	.01	123	205	52	58
	108	(52)	Wyoming	V, S	89	29	33	250+	6.3	115	181	1.031	520	585	99.92	.01	75	84	+.07	125	219	54	61
	109	(56)	Colorado	V	87	26	30	195	4.3	118	100	1.019	530	670	[5] 98.42	.00	74	85	+.10	129	63	50	57
	110	(59)	Texas, Wyoming		87	25	29	205	9.3	116	137	1.028	515	580	99.94	.01	74	85	+.03	125	208	50	57
	111	(52)	Wyoming	V, S	89	31	35	250+	11.5	116	151	1.034	535	590	99.90	.01	76	85	+.12	128	214	51	57
	112	(60)	Midcontinent		86	36	42	125	5.0	123	240	.997	530	620	99.78	.01	75	87	+.06	134	39	57	66
	113	(61)	Midcontinent, Wyoming, Texas.		90	27	30	183	11.8	117	169	1.015	465	540	99.88	.07	76	84	.10	128	210	54	60
	114	(62)	Wyoming	V	94	29	31	190	32.0	114	158	1.030	505	605	99.90	.01	80	85	+.05	125	192	53	56
	115	(72)	Texas, Kansas	P, O	88	21	24	110	4.0	121	263	.988	510	650	99.75	.02	74	84	.07	128	71	57	65
	116	(75)	Wyoming	V	86	21	24	220	14.8	115	98	1.024	490	590	99.50	.02	71	83	.00	124	204	45	52
	117	(79)	do	V, F	86	27	31	250+	19.5	115	158	1.026	385	580	99.88	.15	70	86	.20	127	167	48	56
	118	(91)	do	V, B	82	26	32	250+	36.0	116	164	1.032	505	585	99.92	.02	72	88	+.11	128	250+	50	61
	119	(101)	Colorado	P, F	92	25	27	238	9.0	116	123	1.031	400	450	99.97	.43	72	78	2.18	129	250+	35	38

[1] For States included in each of the Bureau of Public Roads regions, see inside front cover.
[2] Numbers in parentheses indicate code designation of refinery.
[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grade asphalts), P=propane fractionation, and F=fluxing (heavy oils).
[4] Plus values represent gains rather than losses.
[5] Sample No. 42, soluble in CS_2=98.74; sample No. 109, soluble in CS_2=98.86.

Table 2.—Test characteristics of supplemental samples of 85–100 penetration grade asphalts

B.P.R. region [1]	Sample identification [2]	Source of crude	Method of refining [3]	Penetration			Ductility		Softening point	Furol viscosity at 275° F.	Specific gravity at 77° F.	Flash point		Soluble in CCl_4	Standard oven test at 325° F., 5 hours			Thin-film oven test, ⅛-inch film at 325° F., 5 hours				
				100 g., 5 sec. at 77° F.	200 g., 60 sec. at 39.2° F.	Penetration ratio, 39.2°/77° F.	5 cm. per minute at 77° F.	1 cm. per minute at 39.2° F.				Pensky-Martens closed cup	Cleveland open cup		Loss	Penetration of residue		Loss [4]	Tests on residue			
																Value	Percent of original		Softening point	Ductility, 5 cm. per min., 77° F.	Penetration at 77° F.	Percent of original penetration
							Cm.	*Cm.*	° *F.*	*Sec.*		° *F.*	° *F.*	*Pct.*	*Pct.*			*Pct.*	° *F.*	*Cm.*		
Region 1	2a (4)			87	31	36	242	150+	113	145	1.018	360	440	99.94	0.59	67	77	2.08	132	250+	33	38
	9a (77)	Venezuela, Texas	V, S, O	90	35	39	169	23.5	117	210	1.021	410	525	99.92	.22	78	87	.47	127	238	50	56
	9b (77)	do	V, S, O	89	32	36	158	22.3	116	210	1.020	415	525	99.87	.22	76	85	.68	128	213	48	54
	11a (90)	do	V	87	38	44	206	14.8	119	195	1.012	460	565	99.91	.07	76	87	.12	128	250+	55	63
Region 2	13a (1)	Venezuela		86	29	34	250+	89.5	120	240	1.033	480	490	99.92	.07	75	87	.33	131	153	52	60
	14a (3)	do	V, S	91	36	40	180	33.5	119	280	1.034	435	495	99.88	.05	80	88	.81	131	109	52	57
	15a (9)	do	S	86	35	41	230	12.3	120	315	1.039	435	510	99.88	.14	75	87	.70	133	99	47	55
	19a (31)	do	V	94	37	39	239	75.0	115	202	1.018	510	560	99.74	.01	85	90	+.02	141	105	61	65
	19b (31)	do	V	92	30	33	175	40.0	116	165	1.022	535	570	99.76	.00	81	88	+.03	124	242	60	65
	19c (31)	do	V	92	32	35	212	22.5	116	189	1.023	510	565	99.89	.00	81	88	+.06	124	250+	58	63
Region 3	33a (83)	Mississippi	V, S	84	42	50	202	6.5	124	-----	1.004	510	600	99.84	.02	75	89	.04	134	33	58	69
Region 4	36a (1)	Venezuela	V, S	89	38	43	199	29.0	119	254	1.033	475	520	99.91	.05	80	90	.25	130	162	55	62
	47a (59)	Wyoming, Texas	S, O	88	28	32	202	24.0	118	160	1.021	515	575	99.84	.03	74	84	.03	126	190	52	59
	53a (89)	Midcontinent	V, O	92	37	40	171	8.5	119	192	.994	515	620	99.81	.00	84	91	+.04	127	150	65	71
	54a (93)	Wyoming	V	89	26	29	250+	4.0	116	215	1.024	480	590	99.94	.05	75	84	.02	127	202	51	57
Region 5	57a (25)	Kansas	V	95	36	38	153	7.0	120	222	.999	435	665	99.70	.03	82	86	.01	131	62	59	62
	62a (55)	Kansas, Oklahoma, Texas.	V, P, B	91	32	35	207	15.5	116	196	1.006	580	645	99.84	.17	80	88	.28	126	181	57	63
	66a (107)	Kansas	V	96	34	35	152	9.5	119	198	.999	555	620	99.85	.04	85	89	.03	131	105	59	61
Region 6	68a (21)	Texas	V, S, O	87	34	39	166	6.8	120	165	1.004	485	580	99.69	.05	72	83	.08	132	22	50	57
	89a (111)	Arkansas	V	85	28	33	152	8.5	119	241	1.021	575	665	99.71	.00	79	93	.07	125	166	61	72
Region 7	90a (24)	California	V, S	97	35	36	205	14.5	118	200	1.026	410	425	99.88	.21	88	91	1.33	131	65	49	51
	91a (40)	do	V, S, B, O	87	24	28	157	146	112	97	1.016	480	580	99.67	.05	77	89	.00	120	197	57	66
	92a (58)	do	V, S	89	21	24	215	45.0	112	101	1.017	510	540	99.92	.03	77	87	.10	123	250+	50	56
	93a (66)	do	V, S	100	27	27	171	250+	111	84	1.015	505	545	99.85	.03	88	88	.16	119	248	62	62
	95a (99)	do	S	87	27	31	207	11.5	119	206	1.031	450	475	99.88	.14	75	86	.70	132	160	46	53
Region 8	99a (43)	Wyoming	V	84	----	----	189	-----	115	-----	1.030	455	560	99.85	.08	74	88	.14	125	167	52	62
Region 9	107a (50)	New Mexico	V	77	20	26	179	5.5	118	125	1.019	480	645	99.88	.05	64	83	.00	127	230	44	57

[1] For States included in each of the Bureau of Public Roads regions, see inside front cover.
[2] Numbers in parentheses indicate code designation of refinery.
[3] V=vacuum distillation, S=steam distillation, O=blowing (oxidation), B=blending (different grade asphalts), and P=propane fractionation.
[4] Plus values represent gains rather than losses.

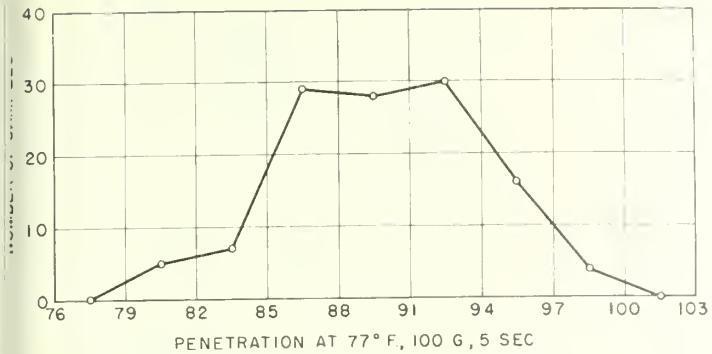

Figure 1.—Distribution of penetration results.

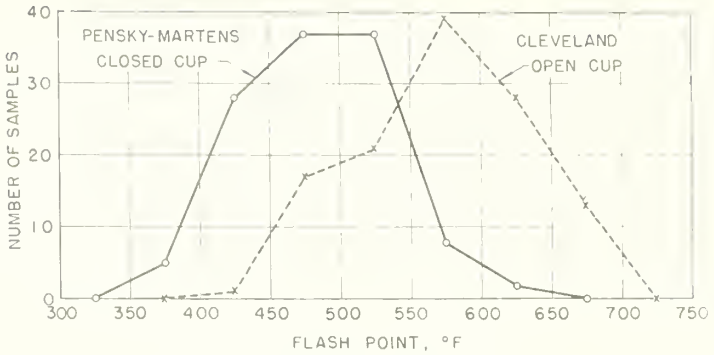

Figure 2.—Distribution of flash point results.

by propane solvent extraction together with arious combinations of distillation, blowing, lending and/or fluxing. Information pertaining to the source of crude used was not iven for 14 asphalts, and information on ie refining method was not furnished for 20 sphalts.

Since the primary purpose of this report s to present the survey results of the characeristics of asphalt cements produced in the Jnited States, it was believed that the large nass of data could best be shown by graphs. 'requency distribution graphs in the form of requency polygons were selected for this urpose. The results of each test made on he 119 samples given in table 1 were grouped nto class intervals. The number of test 'alues in each class interval were then plotted it the midpoint of the respective class inter'al and connected to form the polygon.

The results shown in table 2 are not included in this analysis since they are essentially a duplication of values given in table 1. Their inclusion therefore would have given an improper representation on the basis of the general production of refineries. The results in table 2 are given principally to make the report complete with respect to samples submitted. The numerical identification number for samples corresponds to the number of the replicate sample included in table 1.

Figures 1–15 show the frequency polygons for both the standard specification tests and special tests such as the thin-film oven test, high temperature viscosity, and others that are now being used in specifications by some of the States or other agencies. The frequency polygons for the various tests show considerable variation. Some have a fair amount of symmetry but others show a considerable amount of skewness. The dispersion of results also varies considerably for the different tests.

Results of Tests

The following brief discussion of each frequency distribution polygon points out certain variations in test characteristics and deviations from specification requirements where applicable.

Penetration at 77° F.

The values for the penetration test at 77° F. ranged from 80 to 99 (fig. 1). Although all 119 samples were supposed to conform to the 85–100 penetration grade, 12 asphalts had values below the minimum requirement of 85; none exceeded the upper limit of 100.

Flash point

Figure 2 shows the distribution of flash points determined by both the Cleveland open cup and Pensky-Martens closed cup. As shown in table 1, the flash point determined by the Cleveland open cup ranged from 440° to 680° F. with one sample flashing below 450° F. State specifications have minimum requirements ranging from 347° to 450° F. Thus, only one asphalt which flashed at 440° F. would fail the most restrictive specification.

Some States have replaced the Cleveland open cup method for determining flash point with the Pensky-Martens closed cup method. Specifications now in force require minimum flash points of either 440° or 450° F. Twenty-six of the 119 asphalts flashed at temperatures less than 440° F., and 33 were less than 450° F. As is normally expected, the Pensky-Martens flash point is lower than that obtained in the Cleveland open cup method. However, there is no definite correlation between Pensky-Martens and Cleveland open cup test values. This is illustrated by the shapes of the polygons in figure 2.

Specific gravity at 77/77° F.

The specific gravity of all asphalts ranged from 0.984 to 1.037 (fig. 3). Eleven samples had values less than 1.00. The two lowest

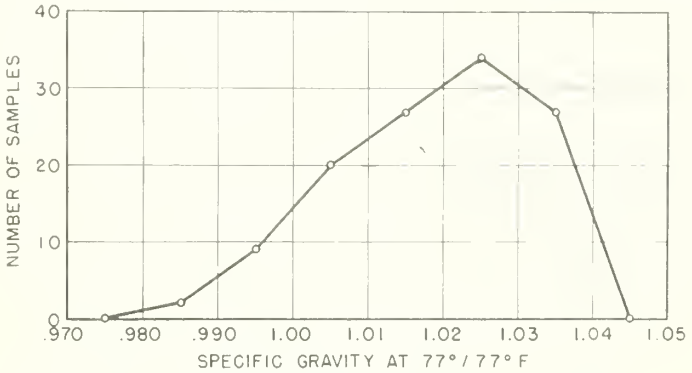

Figure 3.—Distribution of specific gravity results.

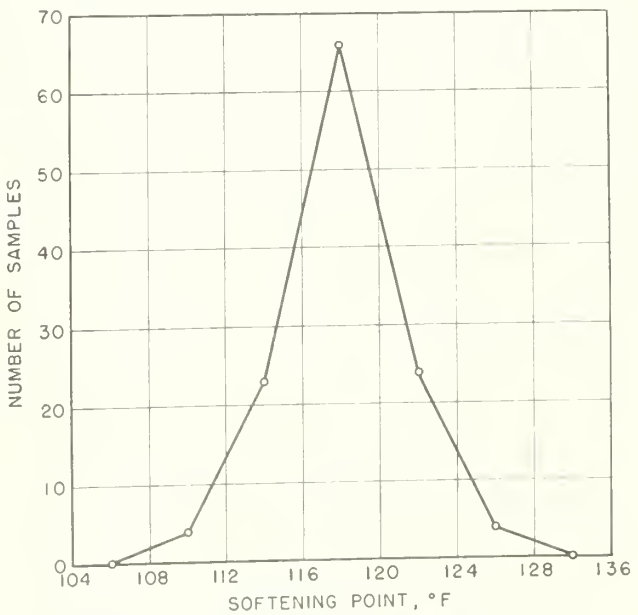

Figure 4.—Distribution of softening point results.

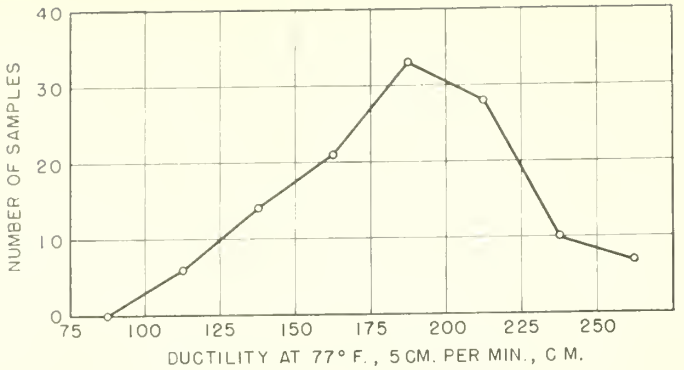

Figure 5.—Distribution of ductility results at 77° F.

values 0.984 and 0.988 were asphalts shipped to different regions from one refinery and were manufactured by a propane method using midcontinent crudes. The peak of the distribution curve is between 1.02 and 1.03.

Softening point

The softening point values for the asphalts ranged from 111° to 125° F., with 66 samples falling in the range of 116° to 119° F. (fig. 4). Only a few States include softening point requirements in their specifications. The most restrictive limits are 100° to 125° F. and 113° to 140° F. Only four asphalts had softening points less than 113° F., and none had values above 125° F.

Ductility at 77° F.

All of the asphalts had ductility values greater than 100 cm., the minimum requirement found in most specifications (fig. 5). Six asphalts had ductilities between 100 and 124 cm., and 14 had ductilities between 125 and 149 cm. Thus, 99 samples had ductilities of 150 cm. or greater, which is the capacity of most machines now in use. Seven samples had ductilities greater than the limit of the Bureau of Public Roads machine which is 250 cm.

Ductility at 39.2° F.

The ductility at 39.2° F., 1 cm. per minute, for the 119 samples reported in table 1 ranged from 3.5 to 250+ cm. (fig. 6). Although low temperature ductility is not widely used, there are specifications which require a ductility of not less than a numerical value of 10 percent of the penetration at 77° F. Forty-three asphalts would fail this requirement. Fifty-five asphalts had values less than 10 cm.

Loss on heating at 325° F., standard test

The change in weight during heating at 325° F. for 5 hours using the standard test ranged from a gain of 0.01 percent to a loss of 0.58 percent (fig. 7). The loss in weight of 115 of the 119 samples was less than 0.20 percent. Most State specifications allow up to 1.0 percent loss. Thus none of the asphalts had loss values even approaching this limit. One State uses a limit of 0.5 percent. Only one asphalt would fail this requirement.

It should be noted that a reduced vertical scale is used in figure 7. Thus, the peak value for this test greatly exceeds that found in the other tests.

Retained penetration of standard oven test residue

The penetration of the residues from the standard loss on heating test ranged from 75 to 94 percent of the original penetration (fig. 8). There were 25 asphalts that retained less than 85 percent, and only 2 of these retained less than 80 percent. Various State specifications include minimum requirements for retained penetration ranging from 50 to 80 percent. Only two materials would fail the most

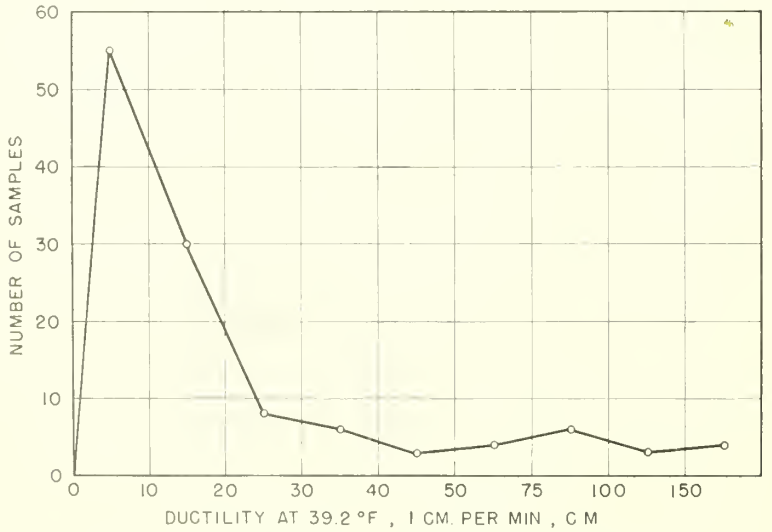

Figure 6.—Distribution of ductility results at 39.2° F.

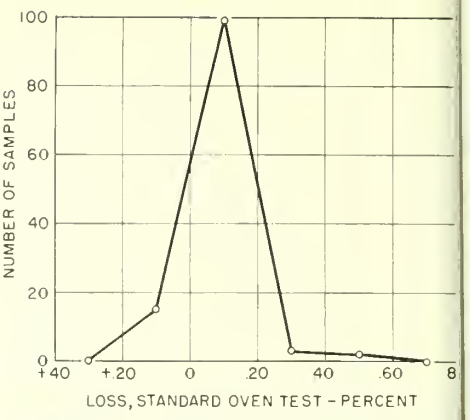

Figure 7.—Distribution of loss during standard oven test.

restrictive specification (in effect in tv States). As in figure 7, the vertical scale reduced in this polygon.

Penetration ratio

The ratio of the penetration at 39.2° 200 g., 60 sec., to the penetration at 77° 100 g., 5 sec., ranged from 22 to 52 (fig. A number of agencies now use this ratio their specifications. The minimum requi ments range from 25 to 35. Eight of the 1 asphalts had penetration-ratio values le than 25, 19 had values less than 30, and had values less than 35. A minimum requi ment of 33 used in one State specificati could not be met by 39 asphalts.

As measured by the penetration ratio, the is a general trend for asphalts from the wes ern regions to have greater temperatu viscosity susceptibility than those from t eastern regions. The central areas general fall in the intermediate range. For examp in Regions 1–3, principally the Eastern Stat only 6 percent (2 out of 35) of the asphal had penetration ratios less than 35 and the had values of 34. For Regions 4–6, prin pally the Central States, 43 percent (23 out 54) of the asphalts had penetration ratios l than 35; and in Regions 7–9, the Weste States, 87 percent (26 out of 30) had rati less than 35. For Regions 4–6, 9 values we less than 30, and 4 were less than 25. F Regions 7–9, 10 values were less than 30, a 3 were less than 25.

Furol viscosity at 275° F.

The Furol viscosity at 275° F. ranged fro 85 to 318 seconds for the 119 asphalts (fig. 10

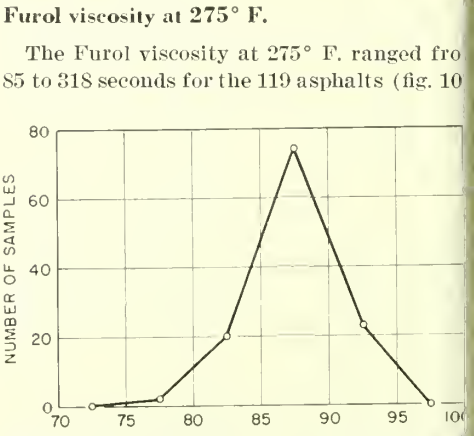

Figure 8.—Distribution of retained penetr tion of standard oven test residues.

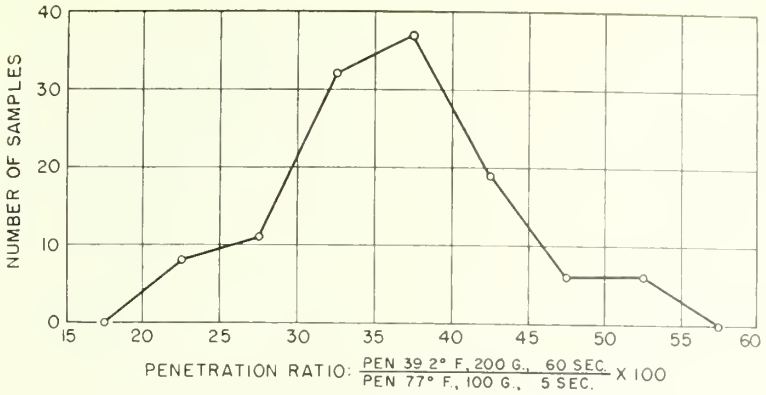

Figure 9.—Distribution of the penetration ratio results.

our asphalts had viscosity values less than 00 seconds, and two asphalts had values exceeding 300 seconds. The viscosity of 81 asphalts was within the range of 150 to 250 seconds, inclusive. Agencies using Furol viscosity as a specification test have specified minimum requirements of 85 seconds. None of the 119 asphalts failed this requirement. The maximum limit of these specifications is 300 seconds. Six asphalts were over this requirement. The general trend for greater viscosity-temperature susceptibility for asphalts in the West as compared with those in the East is also indicated by the generally lower viscosities for asphalts from the Western States.

Thin-film oven test, loss in weight

The change in weight during the heating of the asphalts in the ⅛-inch film for 5 hours at 325° F. ranged from a gain of 0.12 percent to a loss of 2.18 percent (fig. 11). Twenty-seven asphalts gained in weight during heating, 10 asphalts showed no change, and 6 asphalts lost more than 0.50 percent. Specifications now in effect require loss in weight of either not more than 0.75 or not more than 0.85 percent. Only 7 of the 119 asphalts failed the 0.75 percent requirement and 6 of these indicated losses of more than 0.85 percent.

The greater spread of values for the thin-film losses is illustrated in figure 11, the range being approximately four times that for the standard oven losses. This difference also indicates the lack of a definite relation between the loss results for the two tests.

Retained penetration, thin-film residues

The percentage of the original penetration retained by the residues from the thin-film oven test ranged from 38 to 72 (fig. 12). Of the 119 samples, 13 were less than 55 percent and of these 6 were less than 50 percent. Specifications now in effect in several States specify 47 or 50 percent retained penetration. Only 4 asphalts were less than 47 percent.

Comparison of oven tests

A comparison of the distribution of results of retained penetration after the standard and thin-film oven tests is shown in figure 13. This graph has a different grouping from that used in figure 12, in order to show both groups of data on the same basis. The appreciably greater amount of hardening that occurred in the thin-film oven test is indicated. The range in values for the thin-film residues was approximately twice that for the standard test.

Softening point of thin-film residues

The softening point of the residues from the thin-film oven test ranged from 118° to 140° F. (fig. 14). A comparison with the results for the original softening points, which are also shown, indicates the generally higher values and wider spread for the softening point of the thin-film residues. The range in the values of the residues is approximately 1.7 times that of the original materials.

Ductility of thin-film residues

The ductility of the residues from the thin-film test ranged from 13 to 250+ cm. (fig. 15). Initially none of the asphalts had ductility values less than 100 cm. After heating, 22 asphalts were less than 100 cm. and 9 of these were less than 50 cm. There was also a trend for some asphalts to show higher values after the oven test. This was accounted for by the fact that the reduction in penetration or the hardening during heating for these asphalts put them in the range of consistency for optimum ductility. There was much greater dispersion of ductility values for the thin-film residues than for the original asphalts.

Comparison of properties of asphalts

The 85–100 penetration asphalts included in the 1940 and 1941 reports are believed to be generally representative of the asphalt production during the midthirties (*1–2*). A comparison of test properties of those asphalts with the asphalts produced in the mid-

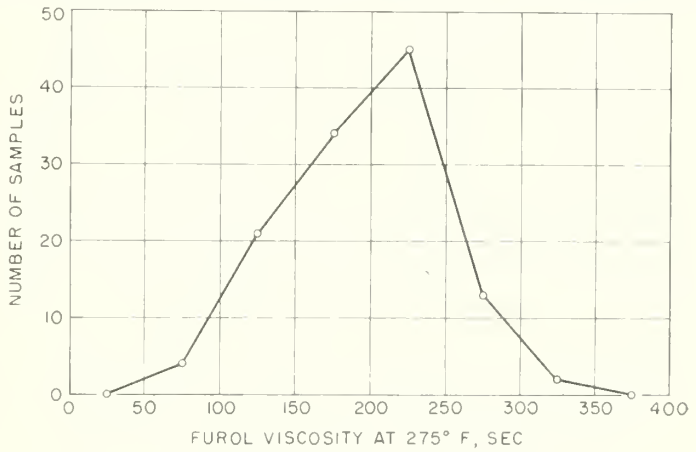

Figure 10.—Distribution of Furol viscosity results.

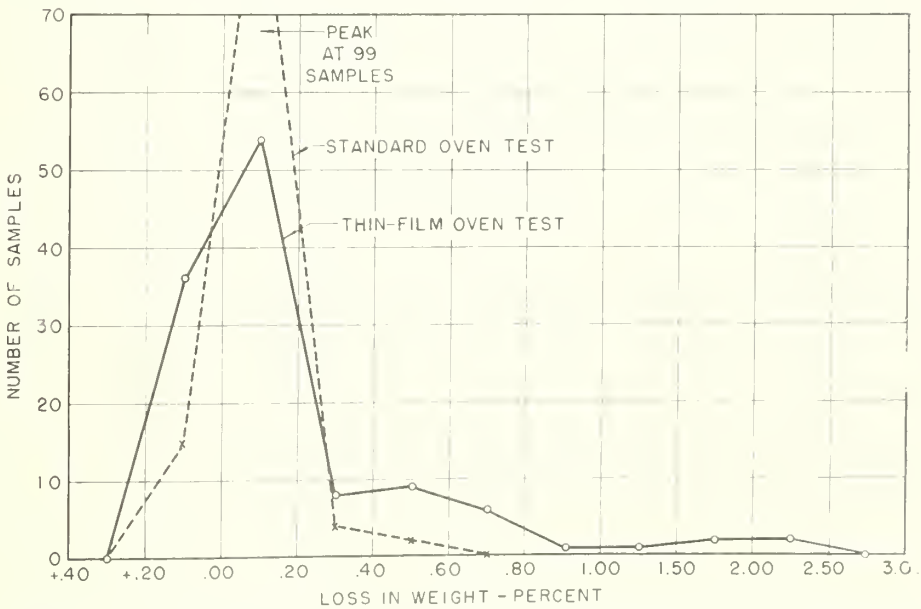

Figure 11.—Distribution of loss in weight for thin-film oven test and comparison with loss in standard test.

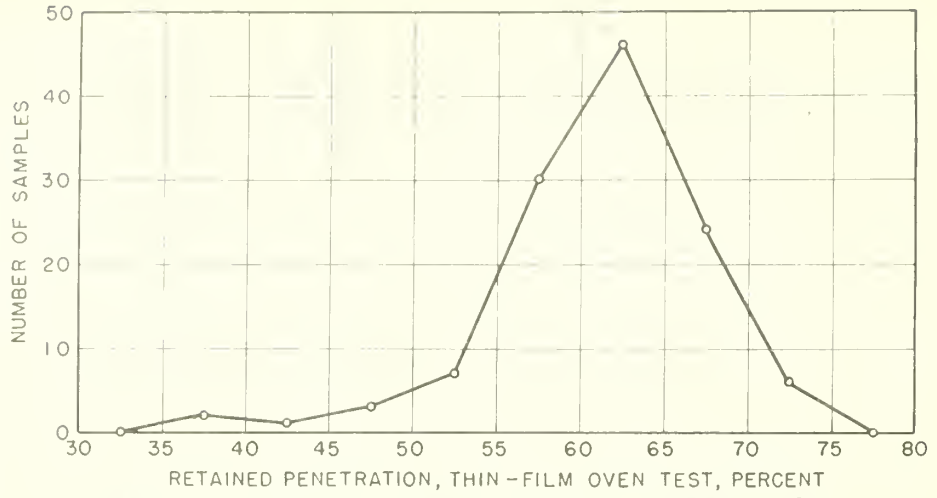

Figure 12.—Distribution of retained penetration of thin-film residues.

fifties should be of interest. Therefore, the results of several tests from the two surveys are presented here in frequency distribution polygons. For discussion purposes the two groups of asphalts are designated as 1935 and 1955 asphalts.

Since there were 40 samples of 1935 asphalts and 119 samples of 1955 asphalts, the results falling in each class interval are shown as a percentage of the total number of samples. Only comparisons are shown for those tests that are being used or considered as measures of quality.

Loss on heating at 325° F., standard test

A comparison of the frequency distribution polygons representing the loss in weight in the standard oven test for the 1935 and 1955 asphalts is shown in figure 16. Except for a slightly larger spread in results for the current asphalts, the distribution for the loss in weight values was essentially the same. In both periods the majority of the samples lost less than 0.20 percent.

Retained penetration of standard oven test residue

A comparison of the results of the penetration retained by the residues from the stand-the standard oven test for the 1935 and 1955 asphalts is shown in figure 17. There was a considerable difference in the amount of retained penetration for the two series of asphalts. Twenty percent of the 40 asphalts in the 1935 series had less than 75 percent retained penetration. None of the 119 asphalts in the 1955 series retained less than 75 percent penetration. The peak of the distribution was between 75 and 80 percent for the 1935 asphalts and between 85 and 90 percent for the 1955 asphalts. This difference resulted chiefly from the general absence of highly cracked asphalts in the 1955 series. Nearly all of those asphalts retaining less than 75 percent of their original penetration were cracked asphalts.

Thin-film oven test

Figure 18 shows the distribution of the results of the change in weight during the thin-film oven test for the asphalts produced in 1935 and 1955. The main differences in the asphalts were the greater percentage of the 1935 materials having loss values between 0.40 and 0.60 percent and the larger percentage of 1955 asphalts having losses greater than 0.80 percent. The secondary peak is caused by five samples, or 12.5 percent of the 1935 asphalts with loss values between 0.40 and 0.60 percent, that were from Mexican crude sources. Only one asphalt known to be from this source was included in the 1955 survey and it had a loss in the same range.

The percentage of penetration retained by the residues from the thin-film oven test on the 1935 and 1955 asphalts is compared in figure 19. In general, the 1955 asphalts were more resistant to hardening in the thin-film test than were those produced in 1935. However, only a few asphalts showed excessive loss in penetration during this heat test.

Comparisons of the distribution of the ductility of the 1935 and 1955 asphalts before and after heating in the thin-film test are shown in figures 20 and 21. The frequency distribution polygons of the asphalts produced for the two periods as shown in figure 20 are essentially the same with a slight trend for the more recent asphalts to have higher ductility. Figure 21 shows a definite trend toward higher ductility for the residues from the 1955 asphalts. Of the 1935 asphalts, 37.5 percent had ductility values less than 100 cm. as compared with approximately 18.5 percent of the 1955 asphalts.

Penetration ratio

The ratio of the penetration at 39.2° F. to the penetration at 77° F. is coming to be

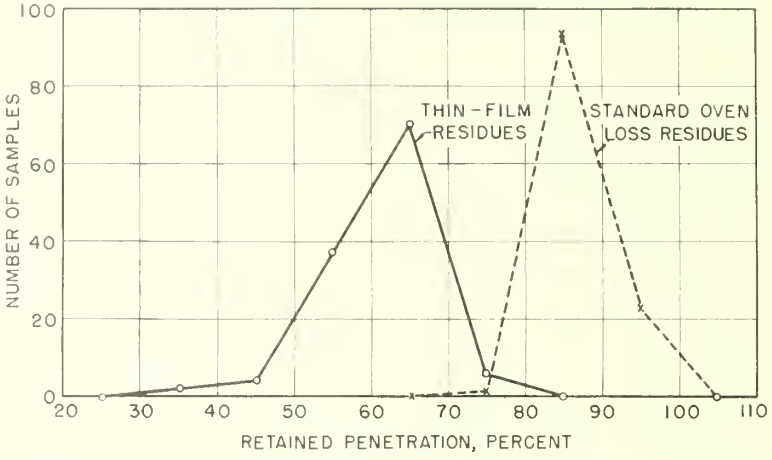

Figure 13.—Comparison of the results of retained penetration after standard and thin-film oven tests.

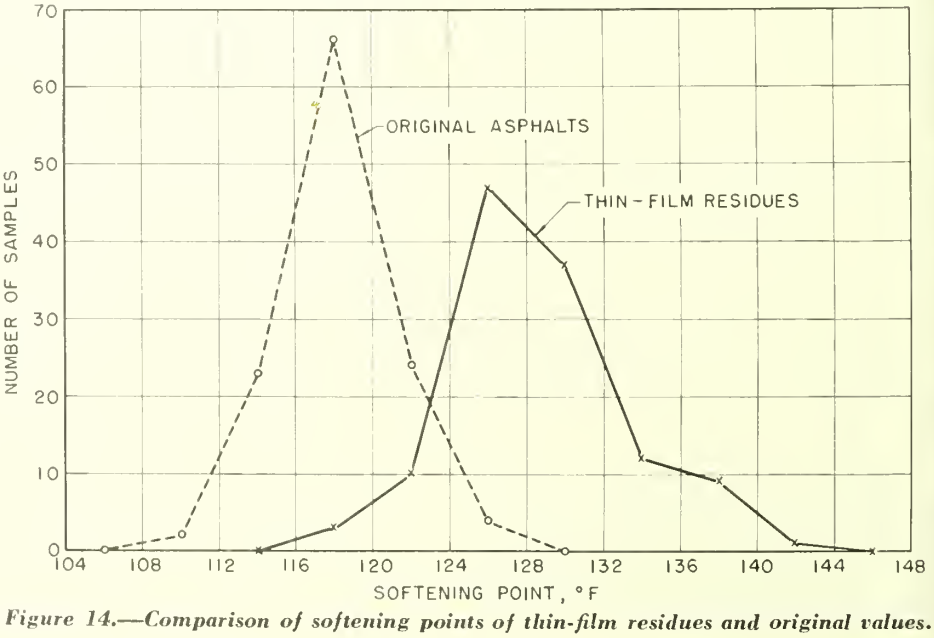

Figure 14.—Comparison of softening points of thin-film residues and original values.

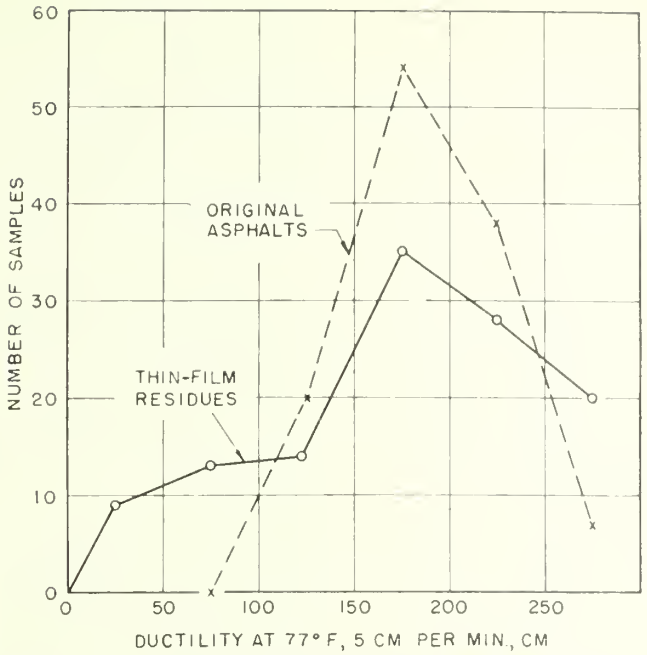

Figure 15.—Comparison of ductility of thin-film residues and original values.

nature generally used as a specification test (fig. 22). None of the asphalts produced in 1935 had penetration ratios less than 25 as compared with 6.7 percent of those produced in 1955. However, 21.1 percent of the 1935 asphalts had values between 25 and 29 as compared with 9.2 percent of the 1955 samples. Here, the larger percentage of 1935 asphalts is due essentially to the results of four California asphalts and two highly cracked asphalts falling in the 25 to 29 range.

Furol viscosity at 275° F.

A comparison of the Furol viscosity at 275° F. of the asphalts produced in 1935 and 1955 is shown in figure 23. In general, the viscosity of the 1955 asphalts was higher than the 1935 asphalts. The secondary peak of the 1935 asphalts, between 50 and 100 seconds, was caused by the four California and two highly cracked asphalts.

Oliensis spot test

In the 1940 report (*1*) it was shown that 15 of the 40 asphalt cements tested had a positive reaction to the Oliensis spot test, and that the degree of heterogeneity as measured by the xylene equivalent was high for some of them, 7 having values higher than 12. The spot test using standard naphtha on the 1955 asphalts indicated negative results for 105 of the 119 samples. The remaining 14 samples showed a positive spot, but most of these had relatively low xylene equivalents as indicated in table 3. Samples 42 and 109 are unusual in that they showed positive spots with 100-percent xylene. However, both of these samples contain unusually high amounts of organic insoluble in carbon disulfide. This insoluble material most likely produces the spot. Sample 69 has a 12–16 xylene equivalent, and this asphalt is known to be refined from a West Texas crude that produces a positive spot asphalt even when vacuum or steam refining is used. One asphalt of the 1935 group was from the same producer and crude source and had a xylene equivalent of 16–20. All of the other positive spot materials had a xylene equivalent less than 12.

Thus it is indicated that very few of the present day asphalts would fail to pass spot test requirements in specifications, particularly if the standard solvent contains some increment of xylene as is often specified.

General Discussion of Data

The test values reported here are considered important as a group in that they provide needed information concerning what may be considered normal values for specific test characteristics, and also what the expected relation between various associated characteristics may be. The general differences in asphalts produced in different areas of the country are also considered to be of interest. It is believed that such information is needed for a proper evaluation of present specification requirements and as a guide to the acceptability of new tests and requirements. The data should also be helpful to those engaged in asphalt research.

From the standpoint of specification requirements it is shown that very few of the 85–100 penetration grade asphalts failed to meet the standard specifications that are used by a large number of States. Although 12 asphalts failed to fall within the proper penetration range, only 3 would fail any of the other standard AASHO requirements. Sample 103 had a Cleveland open cup flash point of 440° F., 10 degrees below the required 450° F. However, this value would be acceptable in all but 17 States. Samples 42 and 109 would fail the requirement for the amount of insoluble matter in carbon tetrachloride in all States using this test. Both samples would also fail the usual requirement for carbon disulfide insoluble.

The asphalt specifications in most general use fail to measure the relative quality of

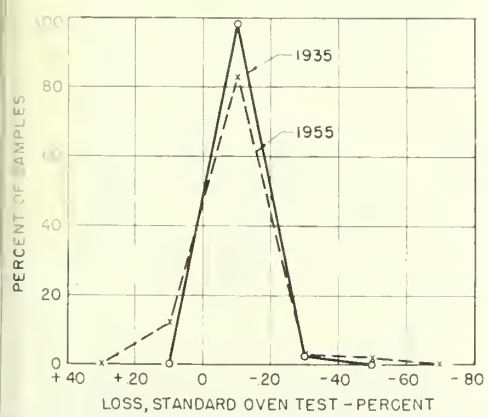

Figure 16.—Distribution of loss in standard oven test, 1935 and 1955.

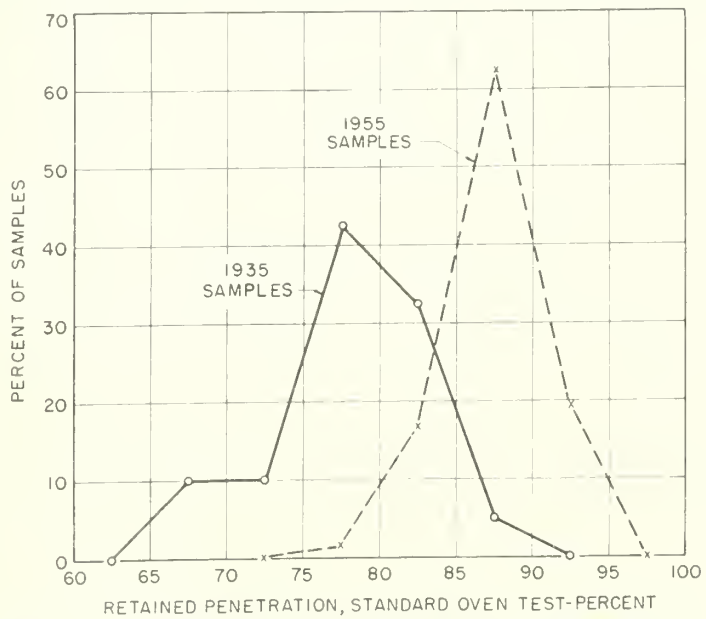

Figure 17.—Distribution of results of retained penetration, 1935 and 1955.

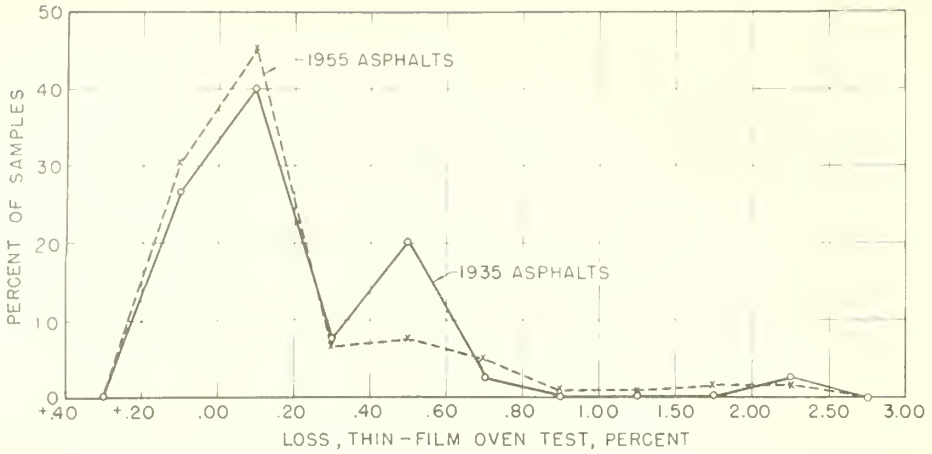

Figure 18.—Distribution of loss in thin-film test, 1935 and 1955.

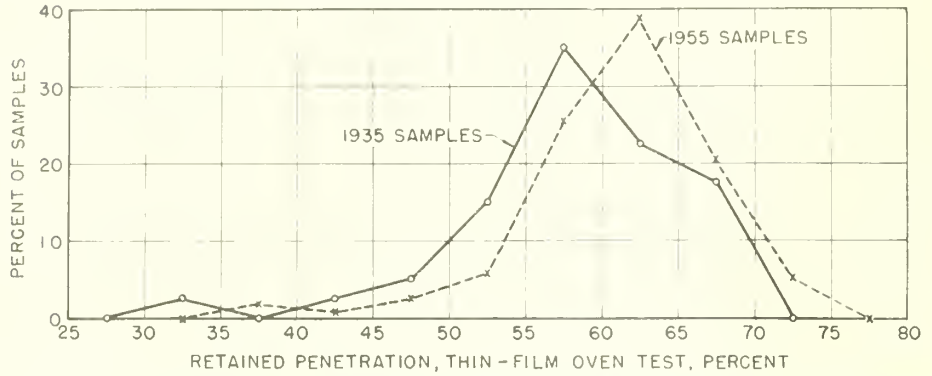

Figure 19.—Distribution of retained penetration after thin-film oven test, 1935 and 1955.

different materials, and some materials meeting present day specifications may actually show poor service characteristics. Consequently, a number of agencies, including the Bureau of Public Roads, are conducting research directed toward the establishment of more direct measures of quality. Several States have adopted special tests designed to raise the overall quality of the asphalts being furnished them. The most extensive effort along these lines has been the adoption, on a trial basis, of a so-called quality specification by a number of Western States. This specification contains several features not usually a part of standard specifications. A general discussion of these requirements with respect to the data reported in this series of tests and the effect of their application on a nationwide basis is believed to be of interest.

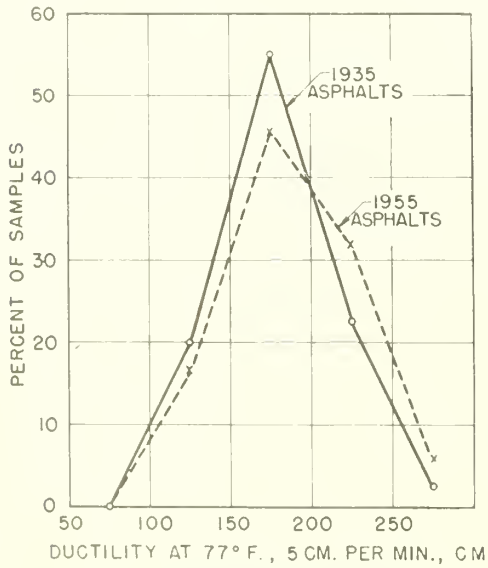

Figure 20.—Distribution of ductility results, 1935 and 1955.

The western quality specification substitutes the Pensky-Martens closed cup apparatus for the Cleveland open cup as the means for determining the flash point. Several States use a limit of 450° F., whereas that recommended by a conference of western producers and consumers in 1957 was 440° F. While such limits may be useful in these States, the tests reported here show that 33 asphalts, most of them of apparently good quality, would fail the requirement of 450° F. if applied on a national basis. The principal reason for including flash point requirements in specifications is to indicate the temperature to which asphalt may be safely heated. Therefore the use of a limit more restrictive than necessary to accomplish this objective can be justified only if correlation with service or laboratory tests for quality is indicated.

Although one State has reported some correlation of Pensky-Martens flash point with service records of asphalts used in that State, such a correlation is apparently not general for asphalts all over the country. These tests show no specific correlation of the flash point values by either method with volatility or the tendency of the material to harden in laboratory tests. There is also no definite relation between the values obtained by the two methods. Apparently certain asphalts contain a small trace of volatile material that is sufficient to give a closed cup flash at a relatively low temperature, but the quantity is insufficient to produce a flash in the open cup apparatus. Losses in weight in both the standard and thin-film oven tests show that some of the asphalts having low Pensky-Martens flash points have relatively low amounts of volatile matter.

A second special test is the penetration ratio, that is, the penetration at 39.2° F., 200 g., 60 sec., divided by the penetration at 77° F., 100 g., 5 sec., multiplied by 100. This test was first proposed for use by an Eastern State with a limiting value of 35. As previously discussed, such a limit would not be applicable on a nationwide basis as it would eliminate almost 90 percent of the asphalts available in the far Western States. The application of the test with a limit of 25, however, serves to eliminate only those materials with unusually high temperature susceptibility, and therefore the test does serve a useful purpose in the western area of the country.

Another feature of the quality specification that serves to limit the viscosity-temperature relation is the Furol viscosity at 275° F. This value is of interest not only as a specification requirement but also as a means of determining the proper mixing temperature. The western specification has limits of 85 to 260 seconds for the 85–100 penetration grade asphalt. None of the asphalts in this series had viscosities lower than 85 seconds, but some of the less susceptible good quality asphalts in the Eastern States had viscosities greater than 260 seconds. Thus again, the same limits could not be applied all over the country.

The substitution of the thin-film oven test (⅛-inch film) for the standard loss test is believed to be a definite step forward. The data in this report show the standard test to be of little or no value. The usual limit of 1 percent loss is unrealistic in that no asphalt being produced today even approaches this limit, even though some of them apparently contain a much greater amount of volatile matter than is consistent with good engineering practice. Of even more importance is the failure of the test to indicate the amount of hardening that is likely to occur in the mixing process.

On the other hand, early investigations

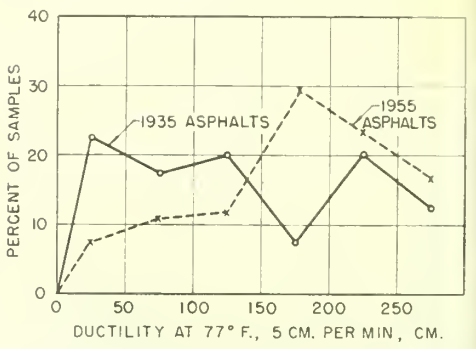

Figure 21.—Distribution of ductility results of thin-film residues, 1935 and 1955.

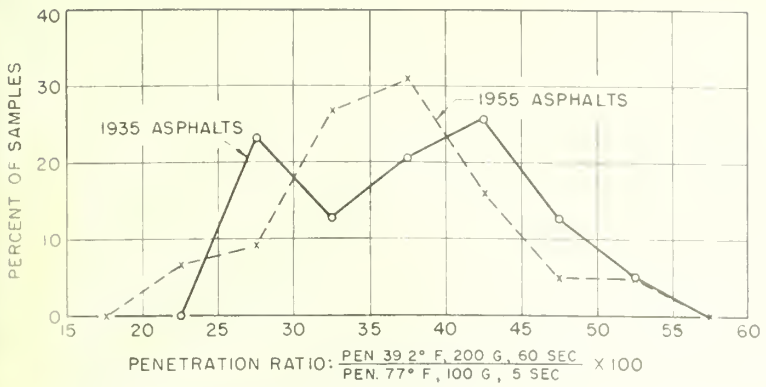

Figure 22.—Distribution of penetration ratios, 1935 and 1955.

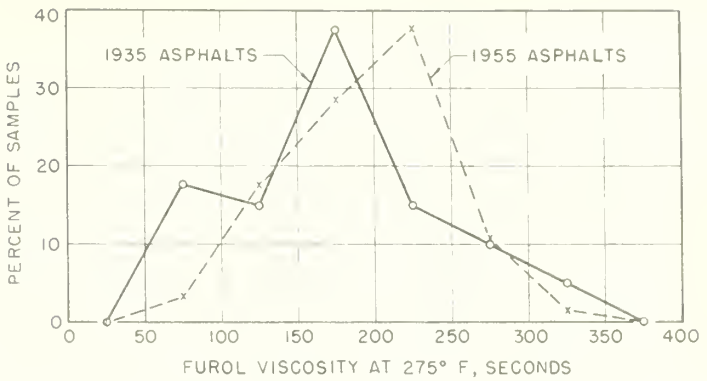

Figure 23.—Distribution of Furol viscosity tests, 1935 and 1955.

have shown that the correlation of the hardening that occurs in the thin-film oven test with the hardening that occurs in normal mixing seems to be generally good for all asphalts. The relation between various tests for hardening was discussed in an article by Pauls and Welborn (4). These authors concluded that a good relation existed between the results of the thin-film oven test and a number of other tests for hardening such as the Shattuck test, an abrasion test on oven-weathered Ottawa sand mixtures, and accelerated outdoor exposure. It was also pointed out that the thin-film oven test is the most suitable for use in specifications because it requires less time, it measures the hardening of the asphalt directly, and it does not require extraction and recovery of the asphalt as in the case of those tests in which an asphalt-aggregate mixture is used. Continuing studies in the Bureau of Public Roads laboratories since that report have not produced any evidence contrary to the conclusions drawn at that time.

Table 3.—Xylene-naphtha equivalents of positive spot asphalts produced in 1955

Sample identification	Xylene-naphtha equivalent
42	[1] 100
43	8–12
56	4–8
59	4–8
69	12–16
78	8–12
90	0–4
95	0–4
98	0–4
102	8–12
103	0–4
109	[1] 100
111	8–12
116	4–8

[1] Positive spot in xylene.

The proper specification limits to be applied to the thin-film oven test to eliminate those asphalts likely to be excessively damaged during mixing without being too restrictive have been the subject of considerable debate. These tests show that the limit of 0.75 percent loss most generally used is not unduly restrictive for any area. The results also indicate that a value of 50 percent used by some States for the amount of retained penetration is also suitable and would be preferable to the 47 percent limit now included in the western quality specification. The minimum value of 75 cm. for ductility as used by the Western States appears suitable for that region. However, this limiting value may be somewhat severe for asphalts from the midcontinent area that normally have lower ductility on the original material. A better evaluation of this limit may be made after the other penetration grades of asphalt collected for this study have been analyzed.

As stated earlier, this is a progress report. The data reported indicate generally the differences in test characteristics that exist and those asphalts that have unusual characteristics. While certain trends and relations are evident, no attempt is made to discuss such relations or to draw definite conclusions at this time. It is believed that the completion of the tests for all grades of asphalts and a survey of the performance of some of the more unusual materials will provide a basis for a better evaluation.

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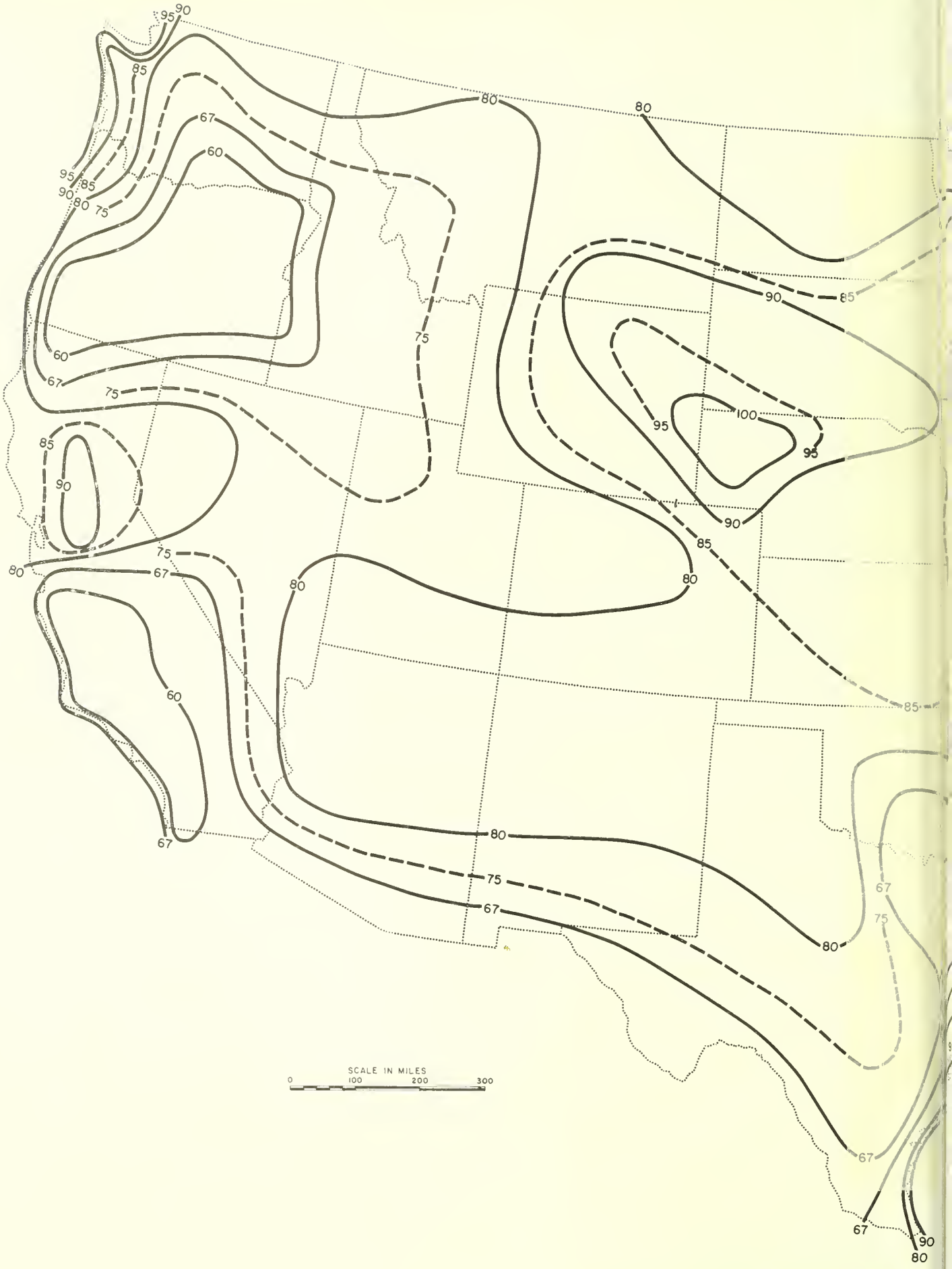
(1) Lewis, R. H., and Welborn, J. Y. *The physical and chemical properties of petroleum asphalts of the 50–60 and 85–100 penetration grades.* Public Roads, vol. 21, No. 1, March 1940; also Proceedings of the Association of Asphalt Paving Technologists, vol. 11, Jan. 1940, pp. 86–157.

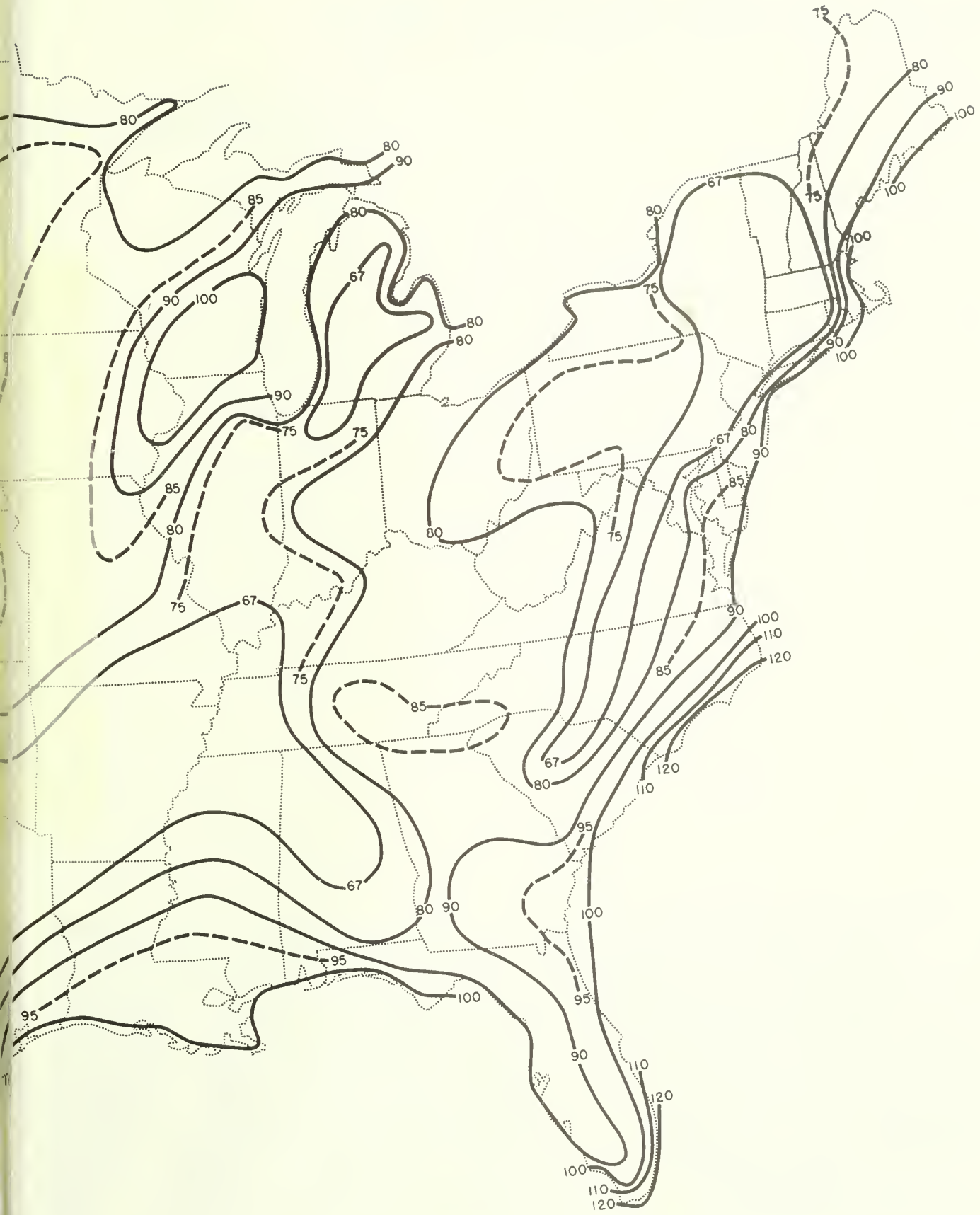
(2) Lewis, R. H., and Welborn, J. Y. *The properties of the residues of 50–60 and 85–100 penetration asphalts from oven tests and exposure.* Public Roads, vol. 22, No. 2, April 1941; also Proceedings of the Association of Asphalt Paving Technologists, vol. 12, Dec. 1940, pp 14–68.

(3) Lewis, R. H., and Halstead, W. J. *Behavior of asphalts in the thin-film oven test.* Public Roads, vol. 24, No. 8, April-May-June 1946.

(4) Pauls, J. T., and Welborn, J. Y. *Studies of the hardening properties of asphaltic materials.* Public Roads, vol. 27, No. 9, Aug. 1953; also Proceedings of the Association of Asphalt Paving Technologists, vol. 21, Jan. 1952, pp. 48–75.



Maximum wind speeds (miles per hour) at 30 fe



ground for 50-year mean recurrence interval.

North Dakota's Use of Aerial Inventory for County General Highway Maps

By E. THOMAS BOWEN, CHARLES J. CRAWFORD, North Dakota State Highway Department, and JOHN B. KEMP, Bureau of Public Roads[1]

North Dakota's topography is particularly adaptable to the aerial method of road inventory. The flat and rolling terrain, the section-line road pattern, and the sparsely populated areas make it possible to quickly identify road and cultural features from the air.

North Dakota, being the first State to institute aerial inventory operations, has in its initial 3 years produced better inventory data in much less time and at less cost than the conventional ground inventory method. Formerly, it was necessary for inventory crews to travel by automobile over every road and street in the State. Since North Dakota is the sixth ranking State in total mileage, the ground inventory method was both costly and time consuming. Under the present inventory method, most of the data can be recorded from the air.

The average inventory costs in 1958, based on the combined use of aerial and ground methods, amounted to approximately 27½ percent of the cost of the ground inventory method used in 1955.

ONE of the principal activities of the State highway planning surveys since their beginning in the midthirties has been inventory and mapping. Probably the principal use of inventory data has been for the preparation of county general highway maps. A number of methods for collecting the inventory data have been developed by the several States.

The North Dakota Highway Planning Survey has instituted an aerial method for collecting most of the field data. During the past 3 years all 53 counties in the State have been inventoried, an area of about 70,000 square miles and including approximately 115,000 miles of roads and streets.

Inventory Operations

The inventory operations in North Dakota involve essentially a combination of the following activities: office preparation of a work map showing roads and cultural data obtained from aerial photographs; verification or revision of the work map based on field observations from the air; estimation and classification from the air of such information as road surface types and widths, and types and sizes of drainage structures; and collection, by ground crews, of structural and other inventory data in incorporated places. Of necessity, certain horizontal control information must also be obtained by a ground crew.

A print of the most current county general highway map, having a scale of 1 inch equals 1 mile, serves as a base for the work map. Information such as road identification numbers, map segment numbers, and other data which serve to orient the air crew are indicated on the base map. Available aerial photographs are then examined for the purpose of updating the base map to the extent possible. The work maps are cut into segments of convenient size for ease in handling within the aircraft.

The work map is then taken aloft and each road is studied. The roads in North Dakota generally follow the north-south or east-west section-line land grids. The road and cultural data that appear on the work map are identified from the air by a check mark on the map. Features not shown on the work map are added by color code, and conversely, features shown on the map but found to be nonexisting are crossed out. Also, incorrect map data are adjusted to conform with observed conditions.

Aerial Classification of Roads

Most of the roads located off the Federal-aid primary system in North Dakota are gravel-surfaced or dirt roads, and for these a satisfactory visual grouping of three width classes has been used: narrow, under 20 feet; middle, 20–26 feet; and wide, over 26 feet. Widths from construction plans are available for practically all surface-treated and higher type roads.

The narrow earth and gravel-surfaced roads under 20 feet are characterized by a single pair of tracks; the middle class, with seldom meeting traffic, is characterized by three tracks; and the over 26-foot widths generally have two pairs of tracks or four clearly defined wheel tracks (fig. 1).

Minor structures, 10 to 20 feet in length, are classified according to size and type from the air. On the basis of test runs, it was found that an experienced air crew could estimate road surface widths and structure size and type information to better than 95-percent accuracy.

Ground Crew Activities

The ground crew and air crew activities are well coordinated to prevent duplication of effort. The aerial inventory is made first,

[1] This article was presented at the 38th Annual Meeting of the Highway Research Board, Washington, D.C., January 1959. Authors' titles are as follows: Mr. Bowen, Assistant Highway Planning Survey Engineer; Mr. Crawford, Highway Planning Survey Engineer; and Mr. Kemp, District Engineer.

Figure 1.—Aerial classification of low type or unsurfaced road widths: (left) narrow class, single pair of tracks; (center) middle class, three tracks; and (right) wide class, two pairs of tracks.

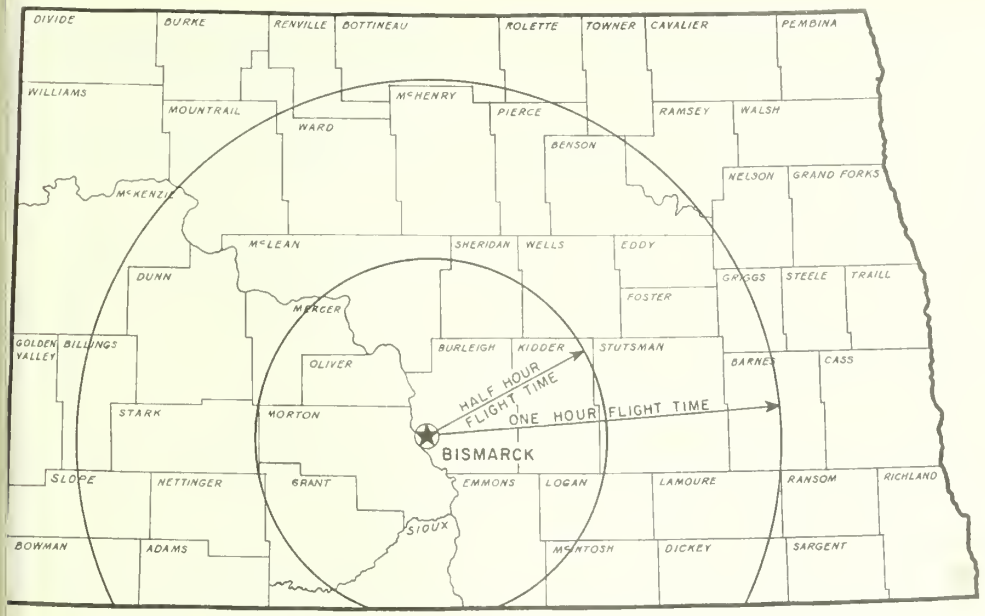


Figure 2.—One-hour flight time from Bismarck covers 58 percent of North Dakota.

nd all possible data are collected by this nethod. The remainder is handled by the round crew.

In areas where there is inadequate coverage f triangulation stations, the ground crew akes the aerial photographs into the field, ocates enough section corners to give ade- uate horizontal control data for mapping, nd pin pricks the section corner locations n the aerial photographs. It should be noted ha the collection of this horizontal control nformation by the ground crew is a one-time pera ion and it has been completed.

For structures with over 20-foot clear span, he ground crew records type, width, length, nd related data. Since most of this infor- nation has been obtained, future reinventory perations will include only the structures uilt, replaced, or destroyed since the pre- ious inventory. Such structures can be eadily identified by the air crew. Inventory ata for structures built on the Federal-aid ystems can be taken from the construction lans.

The air crew has inventoried the smaller nincorporated compacts by the checkoff nethod, working from photocopy enlarge- nents of the aerial photographs. In a few f the larger unincorporated compacts, espe- ially those with over 500 population, some ssistance from the ground crew was needed.

Equipment, Personnel, and Operation

The aerial inventory is accomplished in a our-place aircraft equipped with all-weather nstruments including a directional gyro ompass. The aircraft is leased from a rivately owned corporation exclusively for he inventory operation. The rental rate is 12 per hour without pilot; there is no fixed inimum charge for limited use. A new rplane of this general type so equipped ould cost from $10,000 to $15,000.

The State has full control of all operations nd maintenance. Whenever repairs and aintenance are deemed necessary, whether it be a new motor or a small screw, the State Highway Department initiates the work orders and the corporation pays the bill.

There is strict adherence to FAA safety regulations, both in maintenance and in flight. The corporation carries insurance on damage to the airplane, property damage, and liability coverage for all State and Federal personnel using the craft.

The air crew consists of a pilot and a recorder. The State Highway Department employs a pilot on an annual salary, which includes an increment for flight time. When not engaged in flying activities, particularly during the winter months, he is assigned office duties. A number of State employees hold commercial pilot licenses which qualify them for this type of work. The recorder is one of the regular draftsmen in the Inventory and Mapping Section. Several draftsmen have been trained for this duty.

Three-fourths of the counties in North Dakota are within 1 hour's flight time from the State Capitol in Bismarck (fig. 2), which is the base of operations for the air activities. This saves subsistence and quarters allowances and is convenient in assigning office duties during inclement weather.

In working the outlying counties the air crew collects inventory data in intermediate counties enroute. By the time the inventory of the outlying counties is completed, a major portion of the work in the intermediate counties has been completed also. By proper scheduling of the counties for inventory in any given year, little time is lost in "deadheading".

In rough terrain, such as the badlands, high level reconnaissance at 2,000 to 5,000 feet (above the ground) is necessary for orientation. After completion of the high level reconnaissance, the flight altitude is reduced to less than 1,000 feet for detailed road information.

In flat or gently rolling terrain the high level reconnaissance is employed only in the more congested urban areas. In rural areas the section lines are usually well defined, in many instances with a road or trail. Orientation presents no special problem in such areas and the inventory data can be collected by low level flights. These flights generally are made at an altitude of 200 to 500 feet, and at an air speed of about 80 miles per hour. FAA and North Dakota low flight waivers are required.

The low level county-wide flight runs are made covering one section line at a time and, depending upon the wind direction on a given day, either north-south or east-west section lines are flown. Occasional circling and reruns are necessary if features are not clearly identifiable on the first run. For instance, it may be difficult to ascertain whether a dwelling unit in a grove of trees is occupied. Also, a cluster of houses tends to pass from view too quickly at low altitudes. If the culture is quite dense, it may be necessary to climb to a higher altitude to get a better overall view.

Comparative Costs

As shown in table 1, the area of the average county in North Dakota is 1,324 square miles. During 1955, the last year the ground inventory method was used, the inventory operations in an average county cost $6,104 or $4.61 per square mile. The cost in previous years approximated this amount.

In 1956, the first year of the aerial inventory, the average cost per county was $2,118 or $1.60 per square mile. In 1958, the average costs per county and per square mile were $1,681 and $1.27, respectively. Field inventory costs should be further reduced during subsequent reinventory operations.

Table 1.—Comparison of aerial and ground inventory costs in North Dakota, 1955–58, based on an average of 1,324 square miles per county

Cost basis	1955 (=100) [1]	1956		1957		1958	
		Actual cost	Adjusted cost [1]	Actual cost	Adjusted cost [1]	Actual cost	Adjusted cost [1]
Cost per county:							
Aerial inventory	------	$1,165	$1,095	$966	$899	$807	$751
Ground inventory	$6,104	953	896	914	849	874	813
Total	6,104	2,118	1,991	1,880	1,748	1,681	1,564
Cost per square mile:							
Aerial inventory	------	0.88	0.83	0.73	0.68	0.61	0.57
Ground inventory	4.61	.72	.67	.69	.64	.66	.61
Total	4.61	1.60	1.50	1.42	1.32	1.27	1.18

[1] Cost indexes: 1955=100, 1956=106.4, 1957=107.5, and 1958=107.5. Changes in index values reflect primarily increases in salaries and aircraft operating costs.

Future Operations

Reinventory operations will be continued on a 3-year cycle. This cycle is considered reasonable from the standpoint of efficient inventory and mapping operations, and in view of the need for current data. Under this plan, one-third of the roads and streets would be inventoried each year.

Every effort is being made to transfer ground crew inventory activities to the air crew. In the past, the ground crew has inventoried all incorporated places primarily to identify corporate limits. New methods of obtaining this information by other than a direct visit are under consideration. Also under consideration is the purchase of a camera and appropriate enlarging, developing, and reproduction equipment to permit aerial inventory of areas having relatively heavy cultural development.

Within a short time, it is anticipated that all incorporated places under 500 population, which represent about 70 percent of all incorporated places in North Dakota, will be inventoried by the aerial method without assistance from a ground crew. Ground crew activities will then be limited to inventorying streets in cities over 500 population and to measuring and classifying structures.

Maximum Wind Speeds to Consider in Designing Highway Signs

In cooperation with the Bureau of Public Roads, the U.S. Weather Bureau has prepared the map shown on pages 208–209 which presents wind speeds throughout the United States.

The specific purpose of the map is to provide highway engineers with wind speed values that can be used in the design of highway signs, especially overhead signs. For the first time, the wind data compiled by the Weather Bureau were developed from observations at airport stations instead of locations within cities. A comparison made between city and airport stations in the same locality showed that winds were considerably higher, in most cases, at airports.

Wind speeds shown on the map at an elevation of 30 feet above ground are for the 50-year mean recurrence interval, and take into account thunderstorms and hurricanes. Tornadoes are not subject to prediction by the method used in compiling the map data, and have such extreme wind speeds with a low probability of occurrence that design for their loads is not economically warranted. The map does not show wind direction; hence, the term speed is used instead of velocity.

Adequate guide and destination signs fo the Interstate System have been estimated t cost nearly $1½ billion. The greater dimen sions of the new signs and their positionin along the roadway to provide highway clear ance and legibility make the sign supportin structure an important engineering problem

Highway Statistics, 1957

The Bureau's *Highway Statistics, 1957*, th thirteenth of the bulletin series presentin annual statistical and analytical tables o general interest on the subjects of motor fue motor vehicles, highway-user taxation, Stat and local highway financing, road and stree mileage, and Federal aid for highways, is no available.

The 200-page publication, which include special tables on State legal and administra tive provisions regarding motor-fuel taxatio motor-vehicle registrations, and operator licenses, may be purchased from the Supe intendent of Documents, U.S. Governme Printing Office, Washington 25, D.C., at $1.2 a copy. The series of annual bulletins tha are available from the Superintendent Documents are indicated on the inside bac cover of PUBLIC ROADS.

Characteristics of Traffic Entering and Leaving the Central Business District

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by DAVID A. GORMAN, Highway Research Engineer, and STEDMAN T. HITCHCOCK, Chief, Highway Planning Division

The prosperity of the central business district depends to a great extent upon a transportation system capable of carrying the huge volumes of traffic destined to the central area and upon vehicle storage capacity. Knowledge of the movement of traffic entering and leaving the central business district, including its volume and composition, and the ability to estimate these traffic characteristics are prerequisites for good city planning.

The central business district is but one terminus of many trips in an urbanized area. The other major terminus is outside the central business district in the various zones or sections of the urbanized area. This article discusses only one area of the study of urban traffic, the central business district. With the development of more accurate relationships between population, land use, and automobile and transit trips generated outside the central business districts, more reliable predictions of the volume and distribution of travel from the various zones to the central business district should be possible.

The traffic volume data presented here should be considered carefully before applying to any particular city. The wide variations in numbers of vehicles entering and leaving the central business district of cities within a population group demonstrates the need for precaution in making generalizations, except in the case of averages for population groups. Nevertheless, the data do have considerable value as a measure of traffic that can be expected to enter and leave the central business districts in cities of different sizes.

BETWEEN 1945 and 1955, parking surveys were conducted in the central business districts of 91 cities. The surveys consisted of two phases: parking and traffic. In many of these surveys, because of a limited budget, both phases were studied only between 10 a.m. and 6 p.m., hours when parking and traffic problems were most evident. For this reason and for comparative purposes, all data, except when otherwise noted, are presented for this time period.

The 10 a.m. to 6 p.m. period, although desirable for the study of parking, has its shortcomings from the traffic standpoint because of the omission of the morning peak hours of inbound traffic. Since the 10 a.m. to 6 p.m. period included the daily peak hours of outbound traffic and since late afternoon peaks are usually greater than morning peaks, preference was given in the analysis to the outbound traffic volumes as the basis for comparing traffic volumes among cities. This, however, is not intended to detract from the usefulness of the inbound traffic data shown in the tables. Inbound traffic data can also be used for comparing traffic volumes among cities and population groups.

Vehicles Entering and Leaving the Central Business District

The central business district with its concentration of commercial, business, financial, and governmental activities is one of the principal destinations of traffic in an urbanized area.[1] Table 1 shows the number of vehicles entering and leaving the central business district between 10 a.m. and 6 p.m., and the relation between traffic volumes and urbanized area population for each of the 91 cities. From this table it is possible to compare traffic volumes of one city with another, since the data pertain to the same period of the day and were obtained by the same procedure. Table 2 is a summary showing the averages of these data for cities within each population group.

Five types of traffic entered the central business district: (1) Vehicles passing through without parking, 10 a.m.–6 p.m.; (2) vehicles arriving after 10 a.m. and leaving before 6 p.m.; (3) vehicles arriving before 10 a.m. and leaving after 6 p.m.; (4) vehicles arriving after 10 a.m. and leaving after 6 p.m.; and (5) vehicles arriving before 10 a.m. and leaving before 6 p.m. Vehicles in groups 1 and 2 were counted as they entered and again as they left. Vehicles in group 3 were missed entirely; those in group 4 were counted inbound only; and those in group 5 were counted outbound only.

The ratio of outbound vehicles to inbound vehicles between 10 a.m. and 6 p.m. tended to increase with population size, (tables 1 and 2), which shows that more vehicles of group 5 entered the central business district of the larger cities than vehicles of group 4. This seems to indicate that the proportion of employees in the inbound traffic stream increases with population size. This is significant because of the bearing it has upon the type of parking space that is required in the larger cities in contrast to that which is required in the smaller cities. Employees as a group are long-time parkers, whereas persons on business and shopping trips are generally short-time parkers.

Figure 1 shows the number of vehicles by urbanized area population that left the central business district between 10 a.m. and 6 p.m., and the number per thousand population. According to this graph, the number of vehicles leaving the central business district in proportion to population decreased as cities became larger. The slope of the curve for the number of vehicles leaving the central business district approaches the horizontal for cities of about 800,000 or more population which indicates the probability that further increases in population produce only very minor increases in the number of vehicles leaving the central business district.

It should be noted that the curves apply only to averages for cities within population groups and not to individual cities. Using averages for cities within a population group greatly reduced the influence of factors responsible for the variations of the individual cities. It has not yet been possible to identify all of these factors nor to measure them satisfactorily.

Traffic Volumes Compared on Per Capita Basis

The traffic problem is not confined to large cities. On a per capita basis, the central business districts in the smaller cities are greater traffic magnets than those in the larger cities. Thirteen times as many vehicles per thousand population entered the central business district in the average city of the 10,000–25,000 population group as in the over 1 million population group. This large difference is probably due in part to the fact that the area of central business districts increases very slowly in relation to population. The average city in the over 1 million

[1] *Parking Guide for Cities*, Bureau of Public Roads, Washington, 1956, fig. 31, p. 115.

Table 1.—Passenger cars, trucks, and buses entering and leaving the central business district in 91 cities

Population group and city	Year of study	Urbanized area population, 1950	Vehicles entering CBD, 10 a.m.–6 p.m.		Vehicles leaving CBD, 10 a.m.–6 p.m.		Ratio: vehicles leaving/ vehicles entering	Vehicles leaving CBD during peak ½ hour			Ratio: peak ½ hour/ average ½ hour	Trucks and buses leaving CBD, 10 a.m.–6 p.m.[1]		
			Number	Per 1,000 population	Number	Per 1,000 population		Number	Per 1,000 population	Percentage of all vehicles leaving 10 a.m.–6 p.m.		Number	Per 1,000 population	Percentage of all vehicles leaving 10 a.m.–6 p.m.
5,000–10,000:														
Paris, Ky	1955	6,912	9,716	1,406	9,783	1,415	1.01	934	135	10	1.53	2,116	306	22
Decatur, Ind	1948	7,271	5,918	814	6,134	844	1.04	582	80	9	1.52	1,131	156	18
Seymour, Ind	1948	9,629	10,594	1,100	10,752	1,117	1.01	914	95	9	1.36	2,345	244	22
10,000–25,000:														
Wabash, Ind	1948	10,621	9,961	938	10,549	993	1.06	973	92	9	1.48	1,982	187	19
Coatesville, Pa	1951	13,826	15,087	1,091	15,342	1,110	1.02	1,296	94	8	1.35	2,949	214	19
Lewistown, Pa	1952	13,894	14,271	1,027	14,834	1,068	1.04	1,230	89	8	1.33	2,806	209	19
Hanover, Pa	1954	14,048	17,480	1,244	18,090	1,288	1.03	1,925	137	11	1.70	3,335	237	18
Frankfort, Ind	1948	15,028	11,244	748	11,577	770	1.03	1,096	73	9	1.51	1,891	126	16
Huntington, Ind	1948	15,097	14,431	956	15,007	994	1.04	1,423	94	9	1.52	2,485	165	17
West Chester, Pa	1949	15,168	14,961	986	15,211	1,003	1.02	1,461	96	10	1.54	3,149	208	21
Jeannette, Pa	1953	16,172	8,819	545	9,553	591	1.08	814	50	9	1.36	1,679	104	18
Stevens Point, Wis	1947	16,564	9,755	589	10,213	617	1.05	975	59	10	1.53	1,684	102	16
Carlisle, Pa	1953	16,812	17,495	1,041	17,760	1,056	1.02	1,830	109	10	1.65	3,334	198	19
Greensburg, Pa	1952	16,923	20,613	1,218	21,297	1,258	1.03	2,221	131	10	1.67	4,319	255	20
Chambersburg, Pa	1954	17,212	17,639	1,025	18,297	1,063	1.04	1,860	108	10	1.63	3,630	211	20
Martinsville, Va	1949	17,251	10,987	637	11,299	655	1.03	989	57	9	1.40			
Clovis, N. Mex	1950	17,318	16,312	942	16,749	967	1.03	1,501	87	9	1.43	3,509	203	21
Bradford, Pa	1953	17,354	12,940	746	13,472	776	1.04	1,399	81	10	1.66	2,498	144	19
Monessen, Pa	1953	17,896	9,054	506	9,452	528	1.04	944	53	10	1.60	1,585	89	17
Columbus, Ind	1948	18,370	12,613	687	13,387	729	1.06	1,336	73	10	1.60	2,489	135	19
Portsmouth, N. H	1946	18,830	12,906	685	12,591	669	.98	1,076	57	9	1.37	2,143	114	17
Meadville, Pa	1948	18,972	16,398	864	17,117	902	1.04	1,318	69	8	1.23	2,287	121	13
Oil City, Pa	1954	19,581	14,518	741	15,508	792	1.07	1,686	86	11	1.74	2,507	128	16
Anderson, S.C	1947	19,770	16,854	853	17,267	873	1.02	1,365	69	8	1.26	2,906	147	17
Uniontown, Pa	1950	20,471	21,546	1,053	21,696	1,060	1.01	2,032	99	9	1.50	5,086	248	23
Pottstown, Pa	1949	22,589	20,363	901	20,830	922	1.02	1,674	74	8	1.29	4,214	187	20
Butler, Pa	1951	23,487	20,582	876	21,539	917	1.05	2,042	87	9	1.52	3,987	170	19
Pottsville, Pa	1953	23,640	18,877	799	19,607	829	1.04	1,652	70	8	1.35	3,168	134	16
Walla Walla, Wash	1946	24,102	15,658	650	16,291	676	1.04	1,631	68	10	1.60	3,072	127	19
25,000–50,000:														
New Kensington, Pa	1954	25,146	19,365	770	20,024	796	1.03	2,059	82	10	1.65	3,268	130	16
Roswell, N. Mex	1950	25,738	20,937	813	21,303	828	1.02	2,108	82	10	1.58	4,514	175	21
Washington, Pa	1952	26,280	21,142	804	22,585	859	1.07	2,137	81	9	1.51	3,879	148	17
Lebanon, Pa	1953	28,156	23,291	827	24,372	866	1.05	2,524	90	10	1.66	4,956	176	20
Fond du Lac, Wis	1950	29,936	18,172	607	19,056	637	1.05	2,009	67	11	1.69	3,147	105	17
Biddeford-Saco, Maine	1950	31,160	18,489	593	20,003	642	1.08	1,831	59	9	1.46	3,576	115	18
Reno, Nev	1949	32,497	30,141	928	30,768	947	1.02	3,104	96	10	1.61			
Bristol, Tenn.-Va	1950	32,725	22,022	673	23,145	707	1.05	2,147	66	9	1.49			
Lafayette, La.[2]	1955	33,541	20,202	602	21,526	642	1.07					3,995	119	19
Owensboro, Ky	1955	33,651	20,353	605	21,703	645	1.07	2,333	69	11	1.72	3,664	109	17
Boise, Idaho	1948	34,393	24,605	715	25,781	750	1.05	2,657	77	10	1.65	4,786	139	19
Alexandria, La	1947	34,913	16,363	469	16,849	483	1.03	1,418	41	8	1.35	4,021	115	24
Easton, Pa	1948	35,732	24,445	684	24,513	686	1.00	2,006	56	8	1.31	5,211	146	21
Steubenville, Ohio	1952	35,872	18,107	505	18,640	520	1.03	1,539	43	8	1.32	4,155	116	22
Eugene, Oreg	1952	35,879	37,756	1,052	37,515	1,046	.99	3,908	109	10	1.67			
Independence, Mo	1950	36,963	14,903	404	15,240	412	1.02	1,291	35	8	1.36	2,723	74	18
Norristown, Pa	1949	38,126	27,937	733	28,413	745	1.02	2,573	67	9	1.45	7,170	188	25
Monroe, La	1947	38,572	19,462	505	20,400	529	1.05	1,805	47	9	1.42	4,317	112	21
Kokomo, Ind	1948	38,672	19,547	505	20,192	522	1.03	1,753	45	9	1.39	3,560	92	18
Lake Charles, La	1947	41,272	17,154	416	18,306	444	1.07	1,771	43	10	1.55	3,594	87	20
Newport News, Va	1954	42,358	18,688	441	21,982	519	1.18	3,028	71	14	2.20			
Williamsport, Pa	1954	45,047	21,697	482	23,922	531	1.10	2,336	52	10	1.56	4,018	89	17
Anderson, Ind	1948	46,820	24,412	521	24,424	522	1.00	2,700	58	11	1.77	3,702	79	15
Lynchburg, Va	1948	47,727	17,372	364	19,330	405	1.11	2,085	44	11	1.73			
New Castle, Pa	1952	48,834	25,305	518	27,086	555	1.07	2,303	47	9	1.36	4,961	102	18
50,000–100,000:														
Ogden, Utah	1952	57,112	31,166	546	32,101	562	1.03	3,481	61	11	1.74	4,517	79	14
Lancaster, Pa	1954	76,280	45,729	599	47,974	629	1.05	5,203	68	11	1.74	8,884	116	19
Lexington, Ky	1952	76,497	45,902	600	46,929	613	1.02	4,068	53	9	1.39			
Pawtucket, R.I	1945	81,436	26,197	322	26,728	328	1.02	2,373	29	9	1.42	4,854	60	18
Topeka, Kans	1951	89,104	48,114	540	51,860	582	1.08	6,763	76	13	2.09	8,909	100	17
Albuquerque, N. Mex	1949	96,815	32,359	334	33,976	351	1.05	3,371	35	10	1.59	6,606	68	19
Lincoln, Nebr	1950	99,509	35,741	359	38,761	390	1.08	4,748	48	12	1.96	3,127	31	8
100,000–250,000:														
Portland, Maine	1949	113,499	32,769	289	35,604	314	1.09	3,607	32	10	1.62	8,210	72	23
Corpus Christi, Tex	1947	122,956	26,082	212	26,966	219	1.03	2,520	20	9	1.50	4,544	37	17
New Bedford, Mass	1954	125,495	35,393	282	37,463	299	1.06					5,375	43	14
Gary, Ind	1949	133,911	33,979	254	34,786	260	1.02	3,439	26	10	1.58	5,955	44	17
Evansville, Ind	1949	137,573	32,699	238	35,154	256	1.08	3,753	27	11	1.71	7,213	52	21
Charlotte, N.C	1947	140,930	48,422	344	51,200	363	1.06	5,000	35	10	1.56			
Allentown, Pa	1948	145,145	32,252	222	33,712	232	1.05	2,724	19	8	1.29	6,883	47	20
Knoxville, Tenn	1946	148,166	33,304	225	34,786	235	1.04	2,847	19	8	1.31	6,086	41	17
Reading, Pa	1947	154,931	34,458	222	35,553	229	1.03	2,955	19	8	1.33	8,872	57	25
Chattanooga, Tenn	1947	167,764	35,715	213	37,892	226	1.06	3,567	21	9	1.51	7,264	43	19
Harrisburg, Pa	1946	169,646	34,701	205	36,983	218	1.07	3,275	19	9	1.42	8,394	49	23
Spokane, Wash	1947	176,004	49,383	281	54,982	312	1.11	6,462	37	12	1.88	11,740	67	21
Wichita, Kans	1947	194,047	46,011	237	48,552	250	1.06	5,369	28	11	1.77	7,952	47	16
Tulsa, Okla	1954	206,311	63,469	308	66,819	324	1.05	8,084	39	12	1.94	11,681	57	17
250,000–500,000:														
Richmond, Va	1948	257,995	46,457	180	54,816	212	1.18	5,007	19	9	1.46			
Honolulu, T.H	1947	286,928	48,637	170	52,916	184	1.09	5,410	19	10	1.64	11,356	40	21
Omaha, Nebr	1948	310,291	58,549	189	62,967	203	1.08	7,651	25	12	1.94			
Toledo, Ohio	1947	364,344	60,047	165	64,452	177	1.07	5,554	15	9	1.38	12,387	34	19
Memphis, Tenn	1950	406,034	51,357	126	55,721	137	1.09							
Miami, Fla.[2]	1951	458,647	78,152	170	77,969	170	1.00	6,915	15	9	1.42	12,689	28	16
Louisville, Ky	1951	472,736	79,037	167	83,788	177	1.06	8,285	18	10	1.58			
500,000–1,000,000:														
Atlanta, Ga	1945	507,887	64,885	128	64,982	128	1.00	6,377	13	10	1.57	14,122	28	22
Portland, Oreg	1946	512,643	75,475	147	84,264	164	1.12	9,359	18	11	1.78	17,969	35	21
Dallas, Tex	1950	538,924	87,766	163	96,340	179	1.10	10,840	20	11	1.80	17,988	33	19
Providence, R.I	1945	583,346	55,789	96	57,548	99	1.03	5,296	9	9	1.47	11,061	19	19
Seattle, Wash	1946	621,509	84,833	136	91,505	147	1.08	9,709	16	11	1.70	14,752	24	16
Houston, Tex	1953	700,508	114,328	163	125,255	179	1.10	14,170	20	11	1.81	24,278	35	19
Over 1,000,000:														
Baltimore, Md	1946	1,161,852	82,461	71	86,836	75	1.05	8,817	8	10	1.62	20,595	18	24
Cleveland, Ohio	1951	1,383,599	97,518	70	108,803	79	1.12	13,326	10	12	1.96	21,896	16	20
St. Louis, Mo	1950	1,400,058	76,448	55	91,948	66	1.20	11,229	8	12	1.95	24,079	17	26

[1] Commercial vehicles were not reported separately in 10 cities; and in two cities (Louisville and Lexington, Ky.) data were omitted because taxicabs could not be separated.
[2] Adjusted from 9 a.m.–5 p.m. study period to 10 a.m.–6 p.m. period.

Table 2.—Summary of vehicles entering and leaving the central business districts by population groups

Population group	Number of cities	Average urbanized area population, 1950	Vehicles entering CBD, 10 a.m.–6 p.m.[1]		Vehicles leaving CBD, 10 a.m.–6 p.m.		Ratio: vehicles leaving/ vehicles entering	Vehicles leaving CBD during peak ½ hour			Ratio: peak ½ hour/ average ½ hour	Trucks and buses leaving CBD, 10 a.m.–6 p.m.			
			Average number per city	Per 1,000 population	Average number per city	Per 1,000 population		Average number per city	Per 1,000 population	Percentage of all vehicles leaving 10 a.m.–6 p.m.		Number of cities	Average number per city	Average per 1,000 population	Percentage of all vehicles leaving 10 a.m.–6 p.m.
5,000–10,000	3	7,900	8,700	1,107	8,900	1,125	1.02	810	103	9	1.47	3	1,864	235	21
10,000–25,000	26	17,700	15,100	860	15,600	889	1.04	1,450	83	9	1.49	25	2,908	167	18
25,000–50,000	25	36,000	21,700	621	22,700	650	1.05	2,230	64	10	1.56	20	4,161	121	19
50,000–100,000	7	82,400	37,900	471	39,800	494	1.05	4,290	53	11	1.70	6	6,150	76	16
100,000–250,000	14	152,600	38,500	252	40,700	267	1.06	4,120	26	10	1.57	13	7,690	50	19
250,000–500,000	7	365,300	60,300	167	64,700	180	1.08	6,470	19	10	1.57	3	12,144	34	19
500,000–1,000,000	6	577,500	80,500	139	86,600	149	1.07	9,290	16	11	1.69	6	16,695	29	19
Over 1,000,000	3	1,315,200	85,500	65	95,900	73	1.12	11,120	9	11	1.84	3	22,190	17	23

[1]Excludes vehicles already within the central business district at 10:00 a.m.

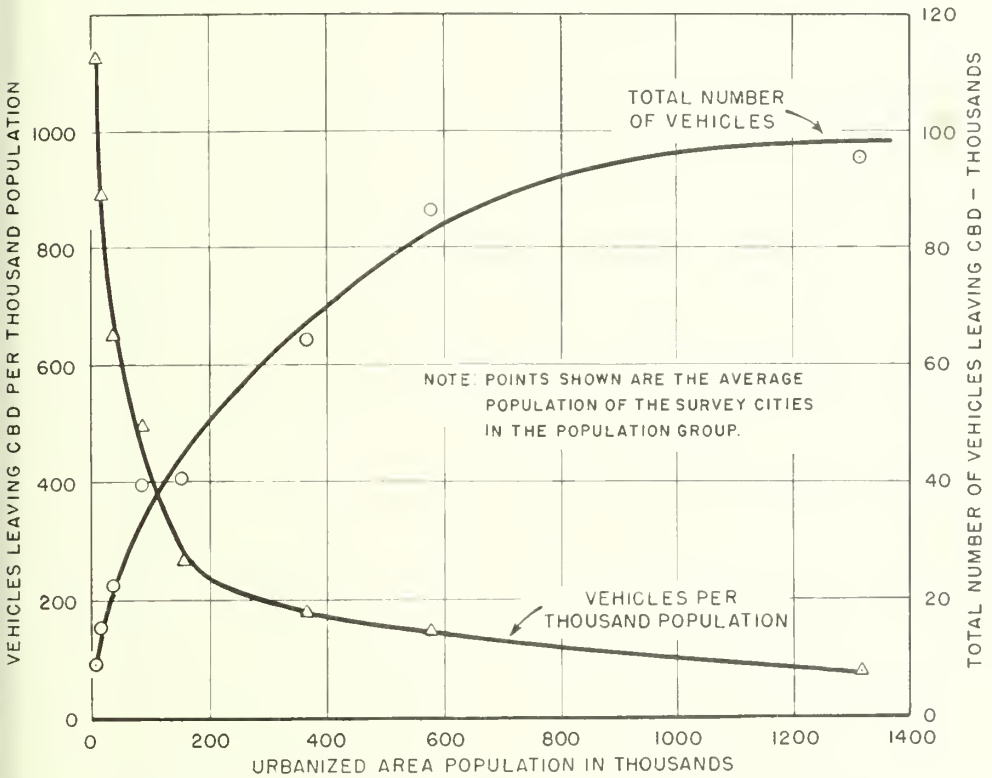


Figure 1.—Vehicles leaving the central business district, 10 a.m.–6 p.m.

population group had only one-ninth as much ground area per capita as the average city in the 10,000–25,000 population group[2]. This does not permit as great a per capita use of the automobile in central business districts of urbanized areas of over 1 million population. The availability of mass transit, the small number of parking spaces in relation to population, the greater proportion of other than downtown destinations in the larger cities[3], and the greater frequency of downtown trips in smaller cities are important contributing factors to the large difference in the per capita ratio of vehicles entering the central business districts of these two population groups. The declining number of vehicles entering the central business districts per thousand population with increasing city size reflects the decreasing relative importance of the central business district in the large urbanized areas as a traffic generator in comparison with total traffic generated in the entire urbanized area.

As shown in table 3, the average number of vehicles entering the central business districts of cities with over 1 million population was only 15 percent greater than the number entering the central business districts of cities in the 500,000–1,000,000 population group. This is in striking contrast to an average population difference of 138 percent. Fewer vehicles entered the central business districts of Baltimore and St. Louis than entered those of Dallas and Seattle, despite the fact that the former are considerably larger cities. The average city of the 1 million or more population group had 74 times the population of the average city in the 10,000–25,000 population group, but only 6 times the number of vehicles entering and leaving between 10 a.m. and 6 p.m. (table 2 and fig. 1).

Variations Among Cities of Same Population Group

The average volume of traffic entering the central business district for the group of cities comprising a population group increased with each progressively larger group, although not in direct proportion. This trend is natural and one that should be expected. The large variations within cities of each population group and the reasons for these variations are probably more significant than the averages for the population groups.

Although population is a very important factor, the variations cannot be fully explained by population alone. Other factors may be just as important in explaining why some cities within the same population group, such as Reno, Nev., and Eugene, Oreg., in the 25,000–50,000 population group and Houston, Tex., in the 500,000–1,000,000 population group, had more vehicles entering the central business district than other cities in the same group. The geographical location of the central business district with respect to the arterial and regional highway system, mass-transit use, economic characteristics of the city, degree of centralization of commercial and industrial facilities, and many other characteristics are all very significant factors that affect the volume of traffic entering the

Table 3.—Comparison of urbanized area populations, areas of central business districts, and vehicles entering CBD in selected cities of the two largest population groups

Comparison	Ratio, population	Ratio, area of CBD	Ratio, vehicles entering CBD
Cleveland/Dallas	2.57	1.75	1.11
Cleveland/Seattle	2.23	1.60	1.15
St. Louis/Dallas	2.60	2.02	.86
St. Louis/Seattle	2.25	1.85	.89
Average city over 1,000,000/average city 500,000–1,000,000 [1]	2.38	1.99	1.15

[1] Cities over 1,000,000 population: Baltimore, Cleveland, and St. Louis. Cities included in 500,000–1,000,000 population group: Atlanta, Portland, Dallas, Providence, and Seattle.

[2] *Parking Guide for Cities.* Table 2, p. 11.

[3] *Parking Guide for Cities.* Figures 5 and 31, pp. 19 and 115.

central business district. These factors are as significant in explaining the variations in traffic volumes of cities within the same population group as they are in accounting for the differences between population groups.

Until the reasons for these variations can be adequately explained and mathematically measured, it probably will not be possible to forecast with confidence the traffic volumes entering central business districts. For example, Monessen, Pa., a coal mining community located along a river, had only 528 vehicles leaving the central business district per 1,000 population during the 8-hour period, whereas Chambersburg, a city having practically the same population in a more highly developed agricultural area of Pennsylvania, had twice as many vehicles leaving the central business district per 1,000 population.

Traffic Volumes Compared on a Square Mile Basis

Vehicle density or the volume of vehicles per square mile of central business district area has an important influence on traffic behavior. Vehicle densities in places where congestion is already acute provide an approximate measure of the maximum number of vehicles that can be accommodated in the central business district under prevailing conditions.

The structural character of the central business district in the old and mature cities, represented by the urbanized areas of 500,000 population and over, crystallized in a period long before the advent of the motor vehicle. Consequently, the central business district inherited a system of streets generally inadequate for the huge volumes of traffic thrust upon it. As a result, although the population continues to grow and the central business district continues to expand both vertically and laterally, there can be only a relatively small increase in the number of vehicles entering the central business district.

Vehicle densities tend to become greater as cities increase in population. Even with a greatly reduced per capita usage of vehicles in the central business districts of the large cities, the vehicle densities in the urbanized areas over 500,000 population were double those in cities under 25,000 population (tables 4 and 5). The greatest density of vehicles in the 91 cities surveyed was 34,000 vehicles per square mile in Dallas, Tex. This included both vehicles parked and in motion.

The peak cordon accumulation (greatest number of vehicles present within the central business district at any time) increased with population size. There were approximately 14 times as many vehicles in the central business district of the average urbanized area of over 1 million population as were found in the average city of 10,000–25,000 population at the moment of peak cordon accumulation (table 5).

Tables 4 and 5 show the peak cordon accumulation divided by the area of the central business district and also by the urbanized area population. Placing all cities on an

Table 4.—Accumulation of vehicles in the central business district in relation to area and population

Population group and city	Urbanized area population, 1950	Area of CBD, square miles	Accumulation of vehicles in CBD, 10 a.m.–6 p.m.			Peak accumulation of vehicles			
			Peak	Average	Ratio of peak to average	Per square mile	Per 1,000 population	Per square mile per 1,000 population	Time of occurrence
5,000–10,000:									
Paris, Ky.	6,912	0.0679	1,023	830	1.23	15,066	148	2,180	4:30
Decatur, Ind.	7,271	.0445	951	636	1.50	21,371	131	2,939	4:30
Seymour, Ind.	9,629	.1280	1,125	844	1.33	8,789	117	913	4:30
10,000–25,000:									
Wabash, Ind.	10,621	------	955	773	1.24	------	90	------	4:00
Coatesville, Pa.	13,826	.1340	1,178	1,094	1.08	8,791	85	636	2:00
Lewistown, Pa.	13,894	.2639	1,650	1,452	1.14	6,252	119	450	3:30
Hanover, Pa.	14,048	.2411	1,889	1,599	1.18	7,835	134	558	3:00
Frankfort, Ind.	15,028	.0559	1,167	904	1.29	20,877	78	1,389	4:00
Huntington, Ind.	15,097	.0773	1,036	944	1.10	13,402	69	888	2:30
West Chester, Pa.	15,168	.1195	1,483	1,267	1.17	12,410	98	818	3:00
Jeannette, Pa.	16,172	.1324	1,589	1,403	1.13	12,002	98	742	3:00
Stevens Point, Wis.	16,564	.0725	934	766	1.22	12,883	56	778	3:30
Carlisle, Pa.	16,812	.3545	2,216	2,038	1.09	6,251	132	372	2:30
Greensburg, Pa.	16,923	.2746	2,562	2,396	1.07	9,330	151	551	2:30
Chambersburg, Pa.	17,212	.2616	1,954	1,766	1.11	7,469	114	434	10:30
Martinsville, Va.	17,251	.1410	1,382	1,194	1.16	9,801	80	568	5:00
Clovis, N. Mex.	17,318	.1167	1,287	1,128	1.14	11,028	74	637	4:00
Bradford, Pa.	17,354	.2498	1,791	1,655	1.08	7,170	103	413	11:00
Monessen, Pa.	17,896	.1197	1,419	1,284	1.11	11,855	79	662	3:00
Columbus, Ind.	18,370	.1025	1,467	1,201	1.22	14,312	80	779	4:00
Portsmouth, N.H.	18,830	.1135	1,704	1,392	1.22	15,013	90	797	5:00
Meadville, Pa.	18,972	.1419	1,557	1,387	1.12	10,973	82	578	4:00
Oil City, Pa.	19,581	.1974	2,160	1,951	1.11	10,942	110	559	2:30
Anderson, S.C.	19,770	.1277	1,778	1,515	1.17	13,923	90	704	11:00
Uniontown, Pa.	20,471	.1948	2,796	2,416	1.16	14,353	137	701	4:00
Pottstown, Pa.	22,589	.2049	1,709	1,560	1.10	8,341	76	369	4:00
Butler, Pa.	23,487	.2219	2,386	2,138	1.12	10,753	102	458	------
Pottsville, Pa.	23,640	.2863	2,140	1,970	1.09	7,475	91	316	10:30
Walla Walla, Wash.	24,102	.1806	1,729	1,501	1.15	9,574	72	397	3:00
25,000–50,000:									
New Kensington, Pa.	25,146	.2583	3,055	2,579	1.18	11,827	121	470	3:00
Roswell, N. Mex.	25,738	.1313	1,703	1,467	1.16	12,970	66	504	5:00
Washington, Pa.	26,280	.3858	2,902	2,537	1.14	7,522	110	286	11:00
Lebanon, Pa.	28,156	.4282	2,632	2,317	1.14	6,147	93	218	11:00
Fond du Lac, Wis.	29,936	.1645	1,888	1,592	1.19	11,477	63	383	3:30
Reno, Nev.	32,497	.3016	5,521	4,970	1.11	18,110	170	557	3:00
Bristol, Tenn.-Va.	32,725	.1848	2,127	1,856	1.15	11,510	65	352	10:00
Owensboro, Ky.	33,651	.1515	2,579	2,271	1.14	17,023	77	506	1:30
Boise, Idaho	34,398	.3214	4,298	3,888	1.11	13,373	125	389	3:30
Alexandria, La.	34,913	.1345	1,819	1,577	1.15	13,524	52	387	2:30
Easton, Pa.	35,732	.1902	2,165	1,873	1.16	11,383	61	319	3:00
Steubenville, Ohio.	35,872	.2164	2,157	1,967	1.10	9,968	60	278	2:00
Eugene, Oreg.	35,879	.3228	5,882	5,173	1.14	18,222	164	508	2:30
Independence, Mo.	36,963	.0807	1,491	1,284	1.16	18,476	40	500	4:30
Norristown, Pa.	38,126	.1628	2,194	2,015	1.09	13,477	58	353	11:00
Monroe, La.	38,572	.1621	2,043	1,763	1.16	12,603	53	327	10:00
Kokomo, Ind.	38,672	.1584	1,808	1,507	1.20	11,414	47	295	4:30
Lake Charles, La.	41,272	.2107	1,980	1,658	1.19	9,397	48	228	10:30
Newport News, Va.	42,358	(1)	[2] 7,548	[2] 6,496	1.16	------	[2] 178	------	4:00
Williamsport, Pa.	45,047	.3476	3,797	3,373	1.13	10,923	84	242	11:00
Anderson, Ind.	46,820	.1492	3,226	2,262	1.43	21,622	69	462	4:30
Lynchburg, Va.	47,727	.2164	3,524	3,154	1.12	16,285	74	341	12:00
New Castle, Pa.	48,834	.1596	3,259	2,834	1.15	20,420	67	418	11:00
50,000–100,000:									
Ogden, Utah	57,112	.2318	3,645	3,330	1.09	15,725	64	275	4:45
Lancaster, Pa.	76,280	.4864	6,077	5,642	1.08	12,494	80	164	2:30
Lexington, Ky.	76,497	.3806	4,776	4,154	1.15	12,549	62	164	3:00
Pawtucket, R.I.	81,436	.1096	2,081	1,842	1.13	18,987	26	233	3:00
Topeka, Kans.	89,104	.4941	6,804	6,048	1.13	13,770	76	155	3:00
Albuquerque, N. Mex.	96,815	.2796	3,689	3,360	1.10	13,194	38	136	11:00
Lincoln, Nebr.	99,509	.4781	7,526	6,859	1.10	15,741	76	158	2:30
100,000–250,000:									
Portland, Maine.	113,499	.3110	7,727	7,155	1.08	24,846	68	219	11:00
Corpus Christi, Tex.	122,956	.3160	4,705	4,374	1.08	14,889	38	121	3:30
New Bedford, Mass.	125,495	------	4,460	3,739	1.19	------	36	------	2:00
Gary, Ind.	133,911	.3299	3,272	2,894	1.13	9,918	24	74	12:30
Evansville, Ind.	137,573	.5287	6,764	6,240	1.08	12,794	49	93	1:30
Charlotte, N.C.	140,930	.3766	7,275	6,773	1.07	19,318	52	137	11:30
Allentown, Pa.	[3] 145,145	.2589	2,401	2,082	1.15	9,274	17	64	2:00
Knoxville, Tenn.	148,166	.380	5,739	5,069	1.13	15,103	39	102	2:00
Reading, Pa.	154,931	.3527	4,831	4,340	1.11	13,697	31	88	3:00
Chattanooga, Tenn.	167,764	.48	7,315	6,621	1.11	15,240	44	91	2:00
Harrisburg, Pa.	169,646	.3666	5,079	4,532	1.12	13,854	30	82	3:00
Spokane, Wash.	176,004	.6345	11,278	10,208	1.10	17,775	64	101	2:00
Wichita, Kans.	194,047	.52	10,850	9,691	1.12	20,865	56	108	3:00
Tulsa, Okla.	206,311	.3301	7,898	7,091	1.11	23,926	38	116	1:30
250,000–500,000:									
Richmond, Va.	257,995	.560	12,372	10,858	1.14	22,093	48	86	11:00
Honolulu, T. H.	286,928	.3392	6,630	5,315	1.25	19,546	23	68	11:30
Omaha, Nebr.	310,291	.5356	11,519	10,632	1.08	21,507	37	69	12:15
Toledo, Ohio.	364,344	.5237	12,865	11,360	1.13	24,566	35	67	2:00
Memphis, Tenn.	406,034	.6303	9,603	8,410	1.14	15,236	24	38	1:00
Miami, Fla.	458,647	.8803	19,414	17,144	1.13	22,054	42	48	3:00
Louisville, Ky.	472,736	.6600	13,057	11,397	1.15	19,783	28	42	1:30
500,000–1,000,000:									
Atlanta, Ga.	507,887	.5406	11,837	10,031	1.18	21,896	23	43	3:30
Portland, Oreg.	512,643	.8956	21,294	19,741	1.08	23,776	42	46	2:00
Dallas, Tex.	538,924	.5905	20,161	18,065	1.12	34,142	37	63	2:00
Providence, R.I.	583,346	.3354	10,856	10,078	1.08	32,367	19	55	2:30
Seattle, Wash.	621,509	.6454	16,041	14,543	1.10	24,854	26	40	2:00
Houston, Tex.	700,508	.7331	22,735	20,126	1.13	31,012	32	44	12:00
Over 1,000,000:									
Baltimore, Md.	1,161,852	.7558	14,172	12,681	1.12	18,751	12	16	2:00
Cleveland, Ohio.	1,383,599	1.0306	28,644	24,707	1.16	27,794	21	20	2:00
St. Louis, Mo.	1,400,058	1.1943	26,636	23,331	1.14	22,303	19	16	1:00

[1] Not available. [2] Influence of U.S. Navy Yard. [3] Estimated.

Table 5.—Summary of accumulation of vehicles in the central business district in relation to population and area

Population group	Number of cities	Average area of CBD, square mile	Peak and average accumulation of vehicles per city in the CBD, 10 a.m.-6 p.m.			Average peak accumulation of vehicles per city in the CBD		
			Peak	Average	Ratio of peak to average	Per square mile	Per 1,000 population	Per square mile per 1,000 population
5,000–10,000	3	0.08	1,030	770	1.35	15,100	132	2,011
10,000–25,000	26	.18	1,690	1,490	1.15	10,900	96	622
25,000–50,000 [1]	22	.22	2,820	2,450	1.16	13,500	80	378
50,000–100,000	7	.35	4,940	4,460	1.11	14,600	60	184
100,000–250,000	14	.40	6,400	5,770	1.11	16,300	42	107
250,000–500,000	7	.59	12,210	10,730	1.15	20,700	34	60
500,000–1,000,000	6	.62	17,150	15,430	1.12	28,000	30	49
Over 1,000,000	3	.99	23,150	20,240	1.14	22,900	17	17

[1] Newport News, Va., excluded.

equivalent area basis permits a more equitable comparison within each population group.

Changes in Supply of Parking Spaces

A study of the changes in supply of parking space in 37 cities revealed that the average annual increase in parking spaces over the 9-year period from 1947 to 1956 was only 2 percent in central business districts of urbanized areas with over 1 million population.[4] All of the gain came from offstreet parking spaces where turnover is relatively low, averaging about 1.6. On the other hand, many curb spaces were eliminated where turnover is relatively large, averaging about 4.4. The net result has been a decrease in available parking space.

The difference in the slopes of the upper curves in figures 1 and 2 for cities between 650,000 and 1,300,000 population is a reflection of the longer parking durations and lower turnover rates in the central business districts of the very large population groups. As urbanized areas increase in population, a greater proportion of motorists park offstreet where the turnover, as previously mentioned, is much less than at the curb. The overall turnover rate in the average central business district of the population group over 1 million is about half that of the population groups under 100,000.[5] This results in a considerably larger buildup of vehicle accumulation in the central business districts of the larger urbanized areas. In the larger cities a greater proportion of all vehicles entering were present at the time of peak accumulation. Approximately 60 percent more storage space was required for every 100 vehicles leaving the central business district of the urbanized areas over 500,000 population as that required by the same number of vehicles leaving the central business district in cities under 100,000 population (tables 6 and 7). An average of 880 vehicles per 100 parking spaces left the central business district in cities under 100,000 population, whereas 550 vehicles per 100 parking spaces left the central business district in cities of over 500,000 population (table 7).

The longer parking durations and the resultant high peak accumulation in cities with over 1 million population have an important bearing on the question of the economic feasibility of providing sufficient parking space for all who desire to park in the large cities. They are also important contributing factors in restricting the number of vehicles that can be accommodated during the day both in storage and in movement.

Outbound Traffic Volumes

The daily peak exodus of vehicles from the central business district usually occurred between 5 and 5:30 p.m. In the urbanized areas of over 1 million population, an average of about 11,000 vehicles left between 5 and 5:30 p.m. (table 2 and fig. 3). The maximum peak half-hour traffic volume observed leaving a central business district was 14,200 vehicles in Houston, Tex.

The outbound traffic volumes for the 91 cities during the peak half-hour of traffic flow exceeded that of the average half-hour between 10 a.m. and 6 p.m. by about 60 percent (table 2). The ratio generally increased with population.

Through Traffic Volumes

The major arterial routes of an urban area almost invariably radiate from the central business district. Between 48 and 63 percent of the drivers that entered between 10 a.m. and 6 p.m. did not stop to park (table 7); many entered only because of the pattern of the highway and street system. The percentages in table 7 probably include some vehicles which had passed through the central business district and were parked just outside, with the central business district as the ultimate destination of the drivers. Also, because of the manner in which observations were made, the percentages included vehicles in service stations and garages being serviced or repaired.

The percentages shown in table 7 were computed without regard to trip origins of parkers because trip origin data were not available in most parking reports. Had trip origins of parkers been taken into account, the resulting percentages of the vehicles that entered and did not stop to park would have been greater than those shown in table 7, although only by a small amount, since some parkers had come from other parking places inside the central business district.

During the peak half-hour of traffic movement, usually between 5 and 5:30 p.m., as many as 98 percent of the vehicles in one of the cities in the 500,000–1,000,000 population group passed through the central business district without parking (table 6). In many cities, a large proportion of this traffic might have been more satisfactorily diverted around

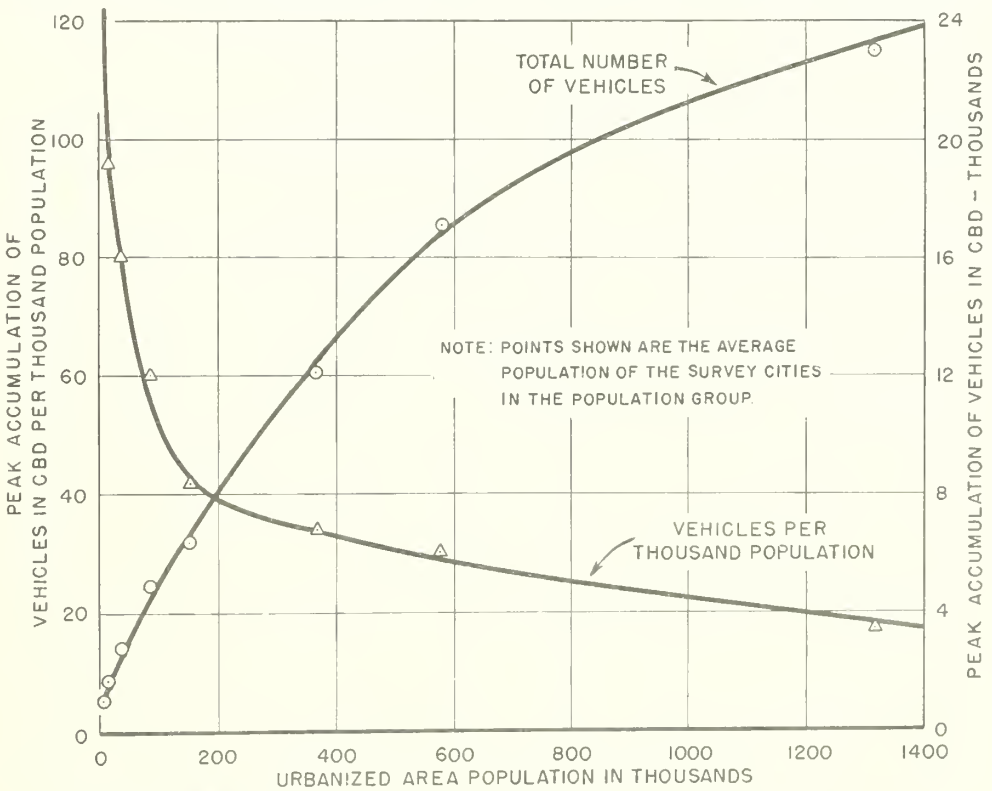


Figure 2.—Maximum accumulation of vehicles in the central business district.

[4] *Changes in supply of parking space,* by David A. Gorman. Highway Research Abstracts, Highway Research Board, vol. 26, No. 6, June 1956, pp. 19–22.

[5] *Parking Guide for Cities.* Table 27, p. 30.

the central business district. The proportion of traffic that entered during the peak half-hour of traffic movement and did not stop to park tended to increase with the size of the city. The development of employment centers in sections of the city other than the central business district creates a large movement of population twice a day going to or coming from work. Much of this movement is across town and through the central business district.

There was little difference in the proportion of traffic passing through the central business district without parking during the business day (10 a.m.–6 p.m.) in cities of different population groups (table 7). This substantiates the need for and the general practice of locating and designing street arterials based on peak-hour volumes of traffic.

Truck and Bus Travel in Central Business Districts

The average number of trucks and buses leaving the central business districts from 10 a.m. to 6 p.m., and the average number per 1,000 population, are shown in table 2 for cities of various sizes. Trucks and buses constituted about 20 percent of all vehicles leaving the central business districts, the range being 16 to 23 percent for the 8 population groups. This relatively narrow range for population groups generally applied to individual cities as well.

The number of trucks and buses leaving the central business district in urbanized areas of over 1 million population was only 7 times as great as the number leaving the central business district in cities of 10,000–25,000 population. This may be compared with a population ratio of 74 to 1 for the same groups.

On a per capita basis, 10 times as many trucks and buses left the central business district in cities of 10,000 to 25,000 population as left the central business districts in urbanized areas of over 1 million population. The declining ratio of trucks and buses per thousand population leaving the central business district with increasing size of city may be attributed to the influence of decentralization of industry and business in the larger cities, and also to the provision of bypass routes for trucks around the central business districts of the larger cities. Generally the street pattern in the smaller cities is such that much of the through truck traffic as well as other types of traffic must pass directly through the central business district.

Trip Purposes

There were many reasons why motorists came to the central business district. About half of them entered because the layout of the street system or highway network compelled them to pass through the central business district on their way to some other destination. Of those who stopped and parked, the trip purposes were as shown in table 8. The percentage distribution of trip purposes varied with population. As population increased, a

Table 6.—Passenger cars, trucks, and buses leaving CBD per 100 parking spaces, ratio peak accumulation to outbound traffic, and percentage of through traffic

Population group and city	Total available parking spaces	Vehicles leaving CBD per 100 parking spaces		Ratio: peak accumulation/peak ½ hour outbound traffic	Percentage of vehicles entering and not stopping to park		Peak accumulation as a percentage of total vehicles leaving CBD
		10a.m.–6 p.m.	Peak ½ hour		10 a.m.–6 p.m.	Peak ½ hour	
5,000–10,000:							
Paris, Ky	960	1,012	97	1.1	67	72	10
Decatur, Ind	624	866	66	1.6	57	68	16
Seymour, Ind	969	1,003	94	1.2	64	68	10
10,000–25,000:							
Wabash, Ind	1,016	1,038	96	1.0	48	54	9
Coatesville, Pa	1,329	1,154	98	.9	63	73	8
Lewistown, Pa	2,343	633	52	1.3	33	41	11
Hanover, Pa	2,830	639	68	1.0	57	73	10
Frankfort, Ind	852	1,208	102	1.3	56	50	10
Huntington, Ind	1,054	1,265	115	.9	59	61	7
West Chester, Pa	1,137	1,338	128	1.0	62	67	10
Jeannette, Pa	1,581	604	51	2.0	35	40	17
Stevens Point, Wis	1,267	806	77	1.0	49	54	9
Carlisle, Pa	3,317	535	55	1.2	37	53	12
Greensburg, Pa	2,663	800	83	1.2	53	69	12
Chambersburg, Pa	2,631	695	71	1.1	55	67	11
Martinsville, Va	1,558	725	63	1.4	59	69	12
Clovis, N. Mex	2,104	796	71	.9	45	49	8
Bradford, Pa	2,185	617	64	1.3	32	45	13
Monessen, Pa	1,444	655	65	1.5	28	33	15
Columbus, Ind	1,274	1,051	105	1.1	48	52	11
Portsmouth, N.H	1,056	1,192	102	1.6	68	76	13
Meadville, Pa	1,987	861	66	1.2	48	54	9
Oil City, Pa	2,558	606	66	1.3	31	45	14
Anderson, S.C	2,137	808	64	1.3	42	42	10
Uniontown, Pa	3,136	692	65	1.4	56	69	13
Pottstown, Pa	2,503	832	67	1.0	55	62	8
Butler, Pa	2,850	756	72	1.2	51	62	11
Pottsville, Pa	2,641	742	63	1.0	58	68	11
Walla Walla, Wash	1,569	1,038	104	1.1	43	61	11
25,000–50,000:							
New Kensington, Pa	2,754	703	75	1.5	51	63	15
Roswell, N. Mex	1,648	1,293	128	.8	46	49	8
Washington, Pa	3,841	588	56	1.4	40	46	13
Lebanon, Pa	3,494	698	72	1.0	42	59	11
Fond du Lac, Wis	2,659	717	76	.9	48	59	10
Biddeford-Saco, Maine	3,060	654	60	-------	50	52	-------
Reno, Nev	4,525	680	69	1.8	41	47	18
Bristol, Tenn.-Va	1,794	1,290	120	1.0	65	70	9
Owensboro, Ky	1,937	1,120	120	1.1	70	78	12
Boise, Idaho	5,199	496	51	1.6	43	50	17
Alexandria, La	1,965	857	72	1.3	59	63	11
Easton, Pa	1,932	1,269	104	1.1	58	64	9
Steubenville, Ohio	2,397	778	64	1.4	43	51	12
Eugene, Oreg	5,779	649	68	1.5	52	61	16
Independence, Mo	1,203	1,267	107	1.2	58	52	10
Norristown, Pa	2,395	1,186	107	.9	68	77	8
Monroe, La	2,243	909	80	1.1	59	65	10
Kokomo, Ind	1,641	1,230	107	1.0	59	62	9
Lake Charles, La	2,214	827	80	1.1	52	68	11
Newport News, Va. [1]	7,426	296	41	2.5	5	8	34
Williamsport, Pa	4,427	540	53	1.6	51	60	16
Anderson, Ind	1,975	1,237	137	1.2	77	90	13
Lynchburg, Va	3,006	643	69	1.7	33	33	18
New Castle, Pa	4,403	615	52	1.4	52	59	12
50,000–100,000:							
Ogden, Utah	4,563	704	76	1.0	50	60	11
Lancaster, Pa	6,918	693	75	1.2	50	65	13
Lexington, Ky	4,739	990	86	1.2	60	70	10
Pawtucket, R.I	1,823	1,466	130	.9	66	79	8
Topeka, Kans	6,901	751	98	1.0	40	57	13
Albuquerque, N. Mex	4,321	786	78	1.1	43	53	11
Lincoln, Nebr	7,402	524	64	1.6	25	37	19
100,000–250,000:							
Portland, Maine	4,654	765	78	2.1	53	64	10
Corpus Christi, Tex	6,355	424	40	1.9	47	59	17
New Bedford, Mass	5,930	632	-------	-------	63	-------	12
Gary, Ind	3,714	937	93	1.0	67	78	9
Evansville, Ind	5,912	595	63	1.8	33	58	19
Charlotte, N.C	7,299	701	69	1.5	65	80	14
Allentown, Pa	3,158	1,068	86	.9	61	74	7
Knoxville, Tenn	4,060	857	70	2.0	75	77	16
Reading, Pa	4,202	846	70	1.6	57	63	14
Chattanooga, Tenn	6,393	593	56	2.1	55	58	19
Harrisburg, Pa	4,765	776	69	1.6	62	76	14
Spokane, Wash	8,726	630	74	1.7	47	56	21
Wichita, Kans	9,613	505	56	2.0	23	37	22
Tulsa, Okla	9,119	733	89	1.0	63	80	12
250,000–500,000:							
Richmond, Va	9,170	598	55	2.5	34	47	23
Honolulu, T.H	6,914	765	78	1.2	62	74	13
Omaha, Nebr	9,793	643	78	1.5	56	64	18
Toledo, Ohio	11,002	586	50	2.3	53	70	20
Memphis, Tenn	11,968	466	-------	-------	59	69	17
Miami, Fla	18,563	420	37	2.8	57	81	25
Louisville, Ky	13,793	607	60	1.6	64	81	16
500,000–1,000,000:							
Atlanta, Ga	11,456	567	56	1.9	69	79	18
Portland, Oreg	11,662	723	80	2.3	30	53	25
Dallas, Tex	17,923	538	60	1.9	51	74	21
Providence, R.I	8,089	711	65	2.0	59	98	19
Seattle, Wash	15,855	577	61	1.7	58	83	18
Houston, Tex	19,226	651	74	1.6	59	74	18
Over 1,000,000:							
Baltimore, Md	12,279	707	72	1.6	64	84	16
Cleveland, Ohio	29,120	374	46	2.1	54	72	26
St. Louis, Mo	29,332	313	38	2.4	38	66	29

[1] Values show influence of U.S. Navy Yard.

Table 7.—Summary of vehicles leaving CBD per 100 parking spaces, ratio of peak accumulation to outbound traffic, and percentage of through traffic

Population group	Number of cities	Average number of available parking spaces	Average number of vehicles leaving CBD per 100 parking spaces		Ratio: peak accumulation/peak ½ hour outbound traffic	Percentage of vehicles entering and not stopping to park		Peak accumulation as a percentage of total vehicles leaving CBD
			10 a.m.–6 p.m.	Peak ½ hour		10 a.m.–6 p.m.	Peak ½ hour	
5,000–10,000	3	850	960	86	1.3	63	69	12
10,000–25,000	26	1,960	849	78	1.2	49	57	11
25,000–50,000	24	3,080	856	82	1.3	51	58	13
50,000–100,000	7	5,240	845	87	1.1	48	60	12
100,000–250,000	14	5,990	719	70	1.6	55	66	15
250,000–500,000	7	11,600	584	60	2.0	55	69	19
500,000–1,000,000	6	14,040	628	66	1.9	54	77	20
Over 1,000,000	3	23,580	465	52	2.0	52	74	24

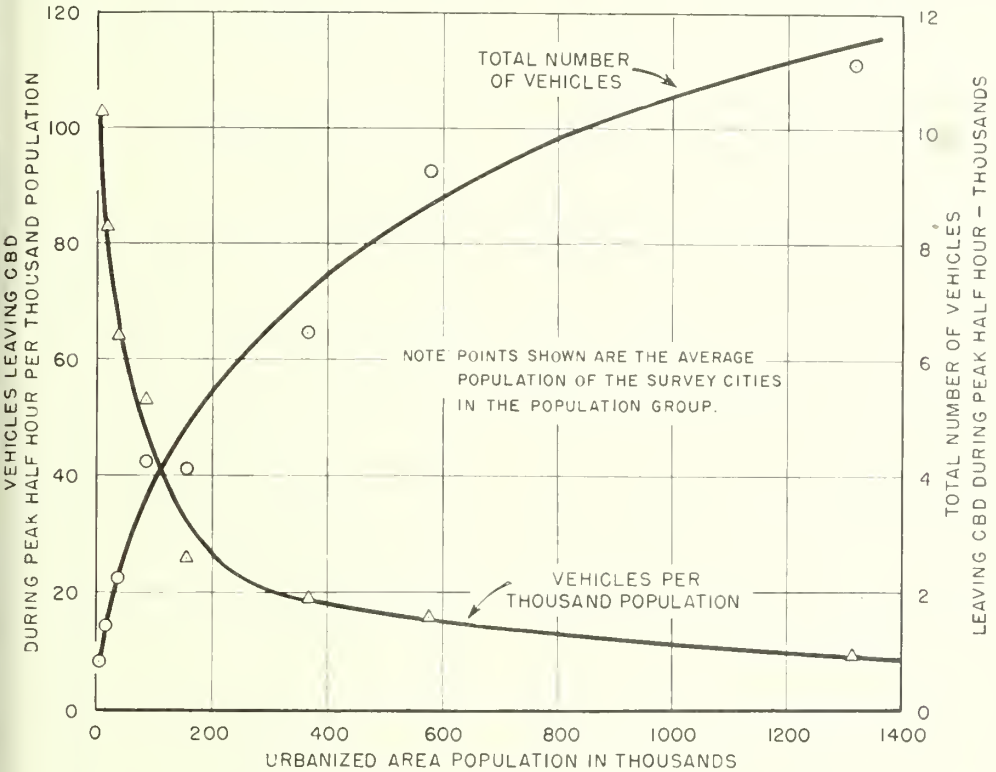


Figure 3.—Vehicles leaving the central business district during peak half hour.

greater proportion of the trips were made for work and a smaller proportion for shopping.

Vehicles in Motion

The wide variations in the number and proportion of vehicles in motion, shown in table 9, limit the usefulness of these data, and the averages are presented here merely for the purpose of shedding some light on the approximate volume and proportion of vehicles in motion within the central business district at an average moment between 10 a.m. and 6 p.m. and also at 5 p.m. The number of vehicles in motion was determined by subtracting the parking accumulation from the cordon accumulation. Disregarding population considerations entirely, the standard deviation of the percentage of vehicles in motion in 87 cities, assuming a normal distribution, was 12 percent around a mean of 25 percent. According to statistical theory, one could expect to find for two-thirds of the time the percentage of vehicles in motion would be between 13 and 37 percent of the cordon accumulation and 95 percent of the time, between 1 and 49 percent.

Generally, the number of vehicles in motion increased as cities became larger. There were 290 vehicles in motion in the average central business district of cities in the 5,000–10,000 population group as compared with 3,210 vehicles in motion in the average central business district of cities in the 500,000–1,000,000 population group. The low figure of 1,910 vehicles in motion in the population group of over 1 million is at variance to the trend and perhaps could indicate a reversal in trend for the largest cities. It may be due to the large percentage of worker-parkers in this population group.

The number and percentage of vehicles in motion at 5 p.m. is also of interest, though of limited value because of the wide variations in the data from which the averages were derived.

Traffic Pattern at Periphery of Central Business District

The cordon traffic data from many cities of various population groups were analyzed in an attempt to discover, if possible, the underlying basic traffic pattern at the periphery of the central business district common to all cities of similar physical characteristics. It is possible that data, such as those presented in table 10 for a 10-hour period (8 a.m. to 6 p.m.), could be useful in forecasting traffic volumes resulting from population growth and in allocating them to a properly designed street system within and around the central business district. They are not adequate for making precise forcasts of traffic volumes and their distribution pattern, and are merely presented as an indication of what can be expected with population increase. Because of the limited information available and the many factors that must be considered, such as type of street, method of operation (whether one-way or two-way), number of lanes, and other traffic controls, these data cannot give the complete answer, but do shed some light upon a most complex subject for which an answer is sought.

The number of streets leading into the central business district is related to the size and geographical location of the central business district. In cities under 25,000 population, there are about 25 street entrances (table 10), whereas cities with over 1 million population have 42 street entrances at the periphery of the central business district.

In cities of 10,000–50,000 population, the street with the greatest inbound traffic volume carried about 19 percent of all vehicles entering the central business district in the 10-hour period between 8 a.m. and 6 p.m. In the larger urbanized areas, a smaller difference among the five most heavily traveled streets was observed in the percentage of

Table 8.—Vehicle trips with known trip purposes, involving parking in the central business district

Population group	Number of cities [1]	Percentage distribution, according to purpose				
		Shopping	Business	Work	Other	Total
5,000–10,000	2	27	32	15	26	100
10,000–25,000	27	31	28	18	23	100
25,000–50,000	23	29	30	18	23	100
50,000–100,000	7	27	31	16	26	100
100,000–250,000	14	25	39	17	19	100
250,000–500,000	8	16	42	21	21	100
500,000–1,000,000	7	17	42	20	21	100
Over 1,000,000	4	12	32	36	20	100

[1] Includes 87 of the cities listed in table 1 and 5 additional cities for which trip purpose data were available.

Table 9.—Average number and percentage of vehicles in motion in the central business district, by population group, during 8-hour period and at 5 p.m.

Population group	8-hour period, 10 a.m.–6 p.m.				5 p.m.		
	Number of cities	Average cordon accumulation	Average number of vehicles in motion	Percentage of average cordon accumulation in motion	Number of cities	Average number of vehicles in motion	Percentage of average cordon accumulation in motion
5,000–10,000	3	770	290	37	3	520	51
10,000–25,000	26	1,510	370	24	23	480	31
25,000–50,000	22	2,450	640	25	18	890	33
50,000–100,000	4	5,000	730	14	4	1,080	21
100,000–250,000	16	5,470	1,580	28	14	1,920	35
250,000–500,000	7	10,730	2,810	26	7	3,810	38
500,000–1,000,000	6	15,430	3,210	24	5	4,370	32
Over 1,000,000	3	20,240	1,910	9	3	2,860	17

Table 10.—Summary of traffic volumes to and from the central business district

Population group [1]	Traffic entering CBD, 8 a.m.–6 p.m.					Traffic leaving CBD, 10 a.m.–6 p.m.				
	Number of cities	Average number of street entrances	Number of streets carrying one-half of traffic	Number of vehicles entering		Number of cities	Average number of street entrances	Number of streets carrying one-half of traffic	Number of vehicles leaving	
				On all streets	Greatest number on 1 street				On all streets	Greatest number on 1 street
10,000–25,000	16	25	4	20,360	4,210	3	21	4	13,410	2,180
25,000–50,000	13	25	4	26,060	4,630	3	20	4	20,180	3,670
50,000–100,000	1	42	6	56,160	6,080	2	32	6	41,980	5,070
100,000–250,000	3	31	6	47,360	5,250	7	27	6	43,330	4,870
250,000–500,000	-----	-----	-----	------	------	2	26	6	74,180	7,820
Over 1,000,000	1	42	7	102,640	12,230	1	41	8	86,840	8,040

[1] No inbound traffic data available for 250,000–500,000 population group. No data available for 500,000–1,000,000 population group.

inbound traffic carried by each street at the central business district cordon.

The largest inbound traffic volume carried on a single street between 8 a.m. and 6 p.m. was 12,230 vehicles which occurred in the population group of over 1 million (table 10). This was 2.9 times as great as the largest inbound traffic volume of 4,210 vehicles entering on a single street in the 10,000–25,000 population group. The population ratio for the two groups was about 74 to 1. In the single city in the population group over 1 million for which data were available, 102,640 vehicles entered between 8 a.m. and 6 p.m. compared with an average of 20,360 vehicles in cities of 10,000–25,000 population, a traffic volume ratio of 4.9 to 1 on only 1.7 times as many street entrances.

Fifteen percent of the streets crossing the central business district cordon carried 50 percent of the inbound vehicles and one-half of the streets carried 90 percent of the inbound vehicles. Of the remaining streets, none carried more than 2 percent of the inbound traffic and many carried less than 1 percent. Similarly, 19 percent of the streets at the central business district cordon carried 50 percent of the outbound vehicles, and one-half of the streets at this cordon carried 85 percent of the outbound vehicles.

These findings, supplemented by a more detailed examination of actual local street conditions, might be especially useful in cities under 50,000 population. Consideration could be given to the feasibility of closing some of the streets to through traffic at the periphery of the central business district and converting them to street parking areas with angle parking. Curb parking might then be entirely eliminated from the few streets with the highest traffic concentration, particularly during the morning and evening rush hours. The increased traffic carrying capacity of these streets would accommodate vehicles displaced from the closed streets.

Since most of the streets at the periphery of the central business district in cities over 100,000 population carry significant volumes of traffic, the closing of any of these streets would be less feasible since a large number of motorists might be inconvenienced.

Application of Data to Emergency Evacuation

Parking and traffic data provide a broad perspective of the scope of the traffic problem involved in evacuating persons and vehicles from the central business districts in an emergency. The data can aid in determining the feasibility of an evacuation plan and in devising various ways and means for its implementation.

As shown in table 6, the ratio of the peak accumulation of passenger cars, trucks, and buses to the peak half-hour outbound traffic volume varied from 0.8 to 2.8, increasing generally with population. Considering only present usage of the street capacity and present traffic-operating conditions, it would require from slightly less than ½ hour to about 1½ hours to evacuate all vehicles from the central business district at the time of the peak vehicle accumulation at a rate equal to the outbound traffic flow occurring daily during the peak half-hour. This evacuation time could obviously be reduced by making most, if not all, streets radiating from the central business district one-way outbound, eliminating crossing movement of traffic and interference from pedestrians, and substituting police control for fixed traffic signals.

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PUBLISHED BIMONTHLY BY THE BUREAU OF PUBLIC ROADS, U.S. DEPARTMENT OF COMMERCE, WASHINGTON

IN THIS ISSUE

The Use of Backwater in the Design of Bridge Waterways 221

Assigning Traffic to a Highway Network . . . 227

Vehicle Acceptance Rates of Parking Areas . . 232

New Publication: *Catalog of Highway Bridge Plans* 233

Traffic Signals and Accidents in Michigan . . . 234

Two Simple Techniques for Determining the Significance of Accident-Reducing Measures . 238

Estimated Travel by Motor Vehicles in the United States, 1957 240

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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 10 *October 1959*

C. M. Billingsley, *Editor*

BUREAU OF PUBLIC ROADS

Washington 25, D.C.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

The Use of Backwater in the Design of Bridge Waterways

BY THE DIVISION OF HYDRAULIC RESEARCH
BUREAU OF PUBLIC ROADS

Reported by JOSEPH N. BRADLEY,
Hydraulic Research Engineer

No generally accepted method has existed for the design of bridge waterways. The determination of the length of a bridge over a stream has been left to the bridge engineer's personal observation and experience. A comparison of the small number of bridge failures to the total number of bridges throughout the country attests to the commendable job bridge designers have performed with the limited design tools available. But, what proportion of these existing bridges are underdesigned or overdesigned from a standpoint of length and clearance? With many new bridges scheduled to be constructed under the accelerated highway program, this question deserves serious contemplation as to safety and economy. Given sufficient time, underdesigned bridges usually speak for themselves. In the case of overdesign, no reliable standards exist at the present time by which these structures can be judged impartially.

Since the subject of bridge waterways is too extensive even for a condensed treatment, the context of this article is confined to a discussion of only one phase of the problem, bridge backwater. It contains a brief account of the problem, the research results, the design information derived therefrom, and the application of bridge backwater to waterway design. The data presented are based on both experimental backwater studies using hydraulic models and field measurements.

IN 1954, a cooperative research project aimed at improving bridge waterway design methods was initiated by the Bureau of Public Roads at Colorado State University at Fort Collins. To date, the investigations have centered on the determination of backwater produced by bridges (*1, 2*),[1] scour at bridge abutments, scour around piers, and methods for alleviating such scour. Two research projects at the University of Iowa, sponsored by the Iowa State Highway Commission and the Bureau of Public Roads, have also contributed needed information on scour at bridge piers (*3*) and scour at bridge abutments (*4*).

Bridge waterway problems are diversified and complex, which account to some extent for the limited progress made in the past in understanding and resolving this phase of design. Because of the many variables involved, hydraulic models were used to serve as the principal research tool in all the work mentioned in the previous paragraph. It is possible with models to hold a certain number of variables constant while investigating the effect of others; then by systematically rotating the combination of variables in the test program, holding some constant and allowing others to vary, it is possible to isolate the part that certain principal variables play in the final result. In addition to aiding in a better understanding of the theory and mechanics involved, the models are indispensable since experimental coefficients are required which can be obtained in no other way.

Experimental Backwater Studies

A comprehensive record of the experimental data, test procedures, and analysis of results on bridge backwater studies appears in a report issued by Colorado State University (*1*). For those interested only in the design application, a booklet entitled *Computation of Backwater Caused by Bridges* (*2*) is recommended. This booklet contains design charts, an explanation of design procedures, and five practical examples. Since the above information is available, it will be necessary to draw from it only sufficiently to understand the contents of this article.

The manner in which flow is contracted in passing through a channel constricted by bridge embankments is illustrated in figure 1. The flow bounded by each pair of streamlines represents 1,000 c.f.s. It will be noted that channel constriction appears to produce very little alteration in the shape of the streamlines near the center of the channel, while a marked change is evident near the abutments where flow from the flood plains enters the constriction. As the discontinuity is greatest in this region, it is apparent that areas adjacent to the abutments can be most vulnerable to attack by scour during floods. Upon leaving the constriction, the flow, which is concentrated in the central portion of the channel, expands at an angle of 5 to 7 degrees on a side until normal conditions are reestablished downstream which may involve a considerable reach of the river.

Constricting the flow of a stream, of course, produces a loss of energy, the greater portion of this occurring in the reexpansion process downstream from the constriction. This loss of energy is reflected in a rise in both the water surface and the energy gradient upstream from the bridge as demonstrated by a profile of this same crossing taken along the centerline of the stream (fig. 2). The normal stage, or water surface existing for a given flood prior to construction of the bridge, is represented by a straight broken line. The water surface for the same flood, with constricting bridge embankments, is denoted by the solid line labeled water surface on centerline (W.S. on ℄). The water surface is above normal stage at section 1, passes through normal stage in the vicinity of section 2, reaches minimum depth near section 3, and returns to normal stage a considerable distance downstream at section 4, where the original regime of the river has not been disturbed. The energy at section 4 is thus the same with or without the bridge. The energy at section 1, on the other hand, must increase to provide head to overcome the loss introduced by the constriction. The major portion of this energy increase is reflected in the backwater, which is the vertical rise in water surface at section 1 (denoted by the symbol h^*_1 in fig. 2).

Note that the drop in water surface measured across the roadway embankment is not the backwater as is so often presupposed to be the case. The water surface as indicated in the central part of the channel at section 3, which is essentially the water surface along the downstream side of the embankments, is invariably lower than normal stage, so the difference in level across the embankments, Δh, is always larger than the backwater h^*_1.

It was found that the backwater to be expected at a bridge for a given discharge is dependent on a number of factors. The more prominent of these are: (1) the degree of constriction of the channel; (2) the number, size, shape, and orientation of piers in the constriction; (3) eccentricity of the bridge with the low-water channel or flood plain; (4) the angle or skew of the bridge with the stream; (5) the type and slope of bridge abutments (important only for the shorter bridges); (6) the amount of scour experienced in the constriction; and (7) the type of crossing; i.e., whether a single bridge or two or more parallel bridges on a divided highway. Contrary to expectations, the

[1] Italic numbers in parentheses refer to the references on p. 231.

width of the abutment or roadway fill had no significant effect on the backwater.

Without a reliable stage-discharge curve for the bridge site, a backwater study can have but limited value. Also, a knowledge of the flood frequency and magnitude is required in order to determine the design discharge for a bridge and the necessary clearance (*2*).

In spite of the number of principal variables just enumerated, the backwater expression and the procedure for computing backwater, as developed from the experimental studies, are very realistic. A person with some training in hydraulics should have no particular difficulty in mastering this phase of waterway design.

An abbreviated form of the expression for computing bridge backwater follows:

$$h^*_1 = K^* \frac{V_{n^2}}{2g} + (\text{- - - - - -}) \qquad \text{Equation (1)}$$

In this expression, K^*, which consists of a combination of experimental backwater coefficients, is multiplied by a velocity head. The overall coefficient K^* varies with the seven geometric factors previously mentioned, while the velocity is computed with respect to the average water cross section under the bridge relative to normal stage. The remainder of the expression, which has been omitted for the sake of simplicity, consists of the change in kinetic energy between sections 1 and 4 (fig. 2) produced by alteration of the stream by the bridge. In many, but not all, of the cases this factor represents a small portion of the total backwater. Guides are provided whereby the importance of this factor can be readily recognized and omitted from the computations where permissible (*2*).

To present a general idea of the manner in which the expression for computing bridge backwater (equation 1) operates, the backwater coefficient for a symmetrical normal stream crossing, having wingwall abutments without piers or other complicating features, may be obtained directly from figure 3. The coefficient K_b (base curve value) varies with the degree of constriction of the channel M and the type of abutment. The parameter M is the ratio of the quantity of flow which can pass through the constriction unimpeded to the total discharge of the river. For no constriction of the stream, $M=1$ and the coefficient is zero. As the degree of constriction increases, M becomes less than unity and the coefficient K_b increases in value. To illustrate, the contraction ratio for the condition shown in figure 1 would be M=8,400/14,000= 0.60. If piers, eccentricity, or skew are involved, the effect of these factors is accounted for by adding incremental coefficients to the value obtained from the base curve which results in an overall coefficient

$$K^* = K_{b\,(\text{base})} + \Delta K_{p\,(\text{piers})} + \Delta K_{e\,(\text{eccentricity})} + \Delta K_{s\,(\text{skew})}.$$

The values of the incremental coefficients for the effect of piers, eccentricity, and skew are obtained from charts prepared for that purpose. A detailed description of the procedure and the charts can be found in reference (*2*).

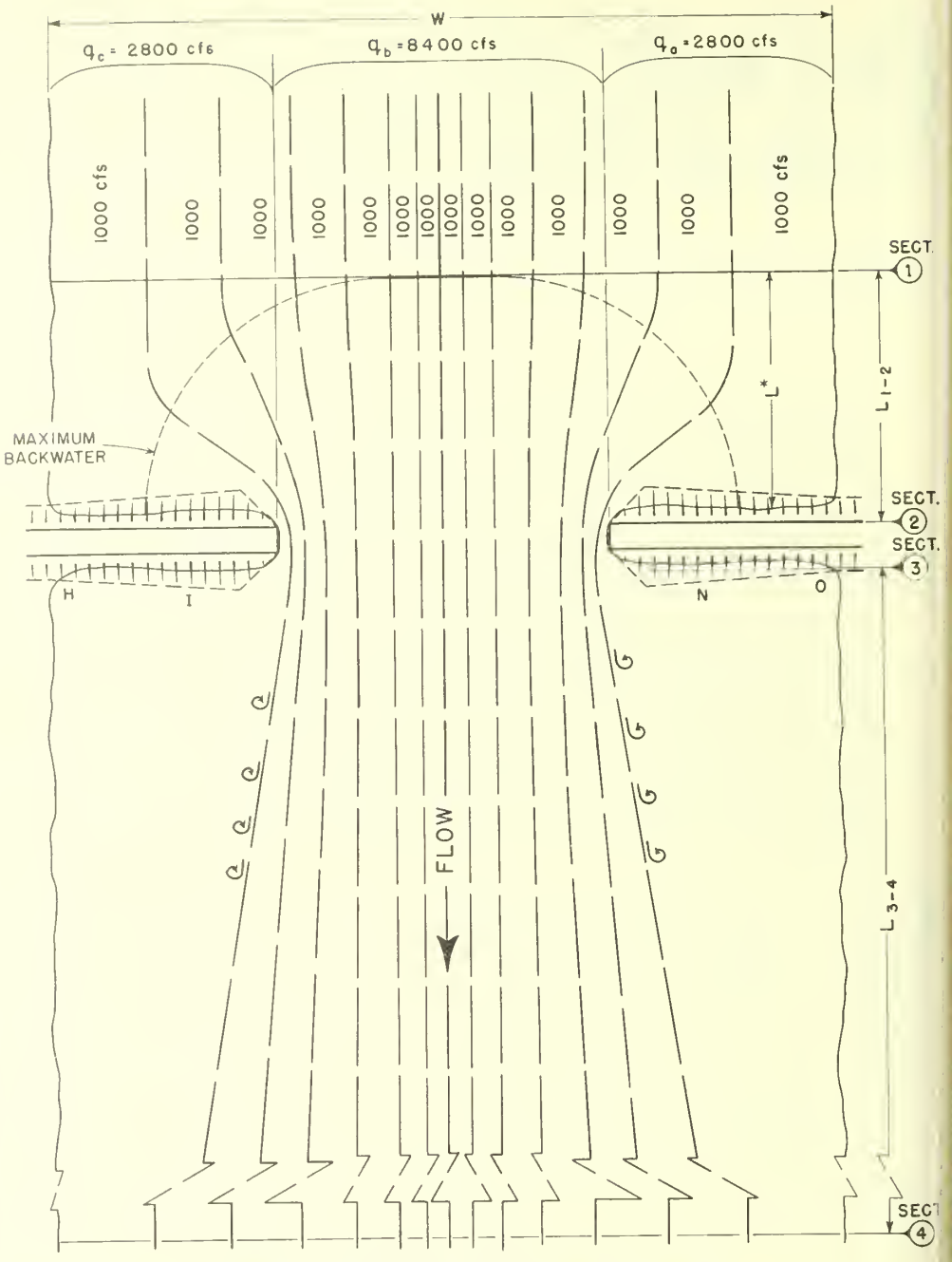


Figure 1.—Flow lines of typical normal crossing.

Reliability of Model Results

Since hydraulic models of bridges have definite limitations, it was imperative that some means be devised to verify or disprove the validity of the experimental information. This was accomplished by applying the computational procedure, developed from the model studies, to existing bridges for which the U.S. Geological Survey had furnished field measurements obtained during floods. Reliable measurements on bridge backwater are extremely difficult to make in the field, but the drop in water surface across embankments, $\triangle h$, is readily measurable (fig. 2). Model results showed a very definite relation existing between the drop in water surface across the embankments, $\triangle h$, and the backwater, h^*_1, so model computations and prototype measurements are compared on the basis of $\triangle h$.

A comparison of measured and computed values for several bridges varying from 20 to 340 feet in length is presented in table 1. Columns 2 through 6 give the bridge length, flood discharge, average velocity under the bridge, the contraction ratio, and the computed backwater, respectively. The computed and measured values of $\triangle h$ are shown in columns 7 and 8, while the percentage difference in each case is shown in column 9. The differences range from −8.5 to +13 percent, the deviation being positive in six instances and negative for six with an average deviation of +2 percent. The deviation in the majority of the cases is well within the error of field measurement. The experimental error of the model experiments is estimated

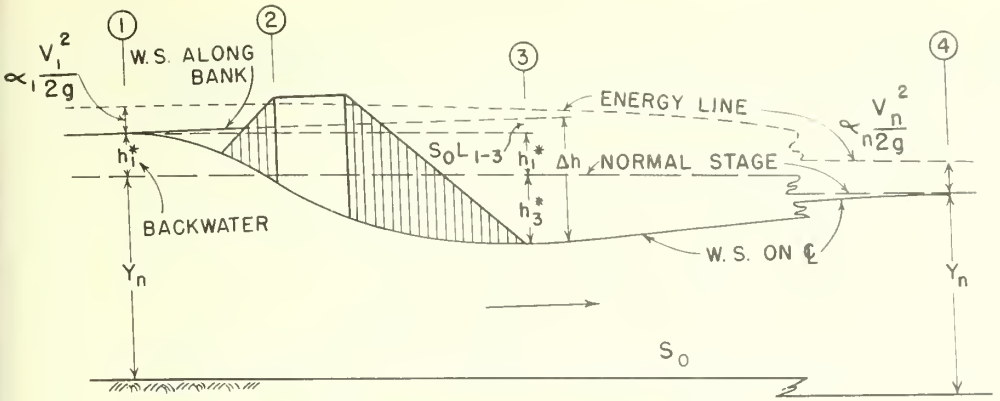


Figure 2.—Profile on centerline of stream.

comparable to the average deviation. Thus, the comparison affords a satisfactory verification.

Application of Backwater to Design

Now that it is possible to compute bridge backwater with a fair degree of confidence, to what practical purpose can this information be used in design?

1. It makes it possible to proportion bridges to operate during floodflows at a limited specified backwater.

2. It offers a fair means of settling claims involved in backwater damage suits instigated by upstream property owners.

3. It makes it possible to understand and compute the hydraulics involved in cases where approach roadways can be overtopped during infrequent floods.

4. It provides a large share of the necessary hydraulic information for a proposed economic analysis to determine the optimum design discharge and the most economical length of bridge.

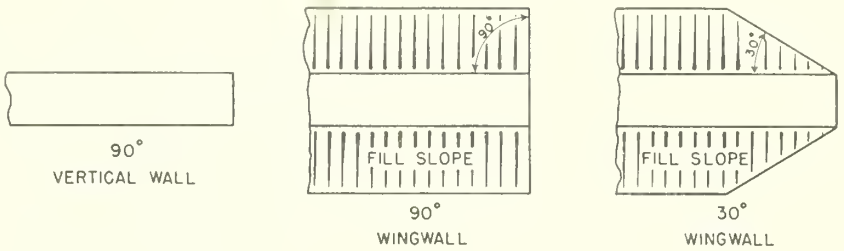
In the case of item 2, no acceptable method has existed for computing backwater produced by bridges. Backwater based on field measurements made by the novice was also justifiably questionable. Thus, damage suits of this nature have resulted in indefinite delays or settlements have been made on considerations other than fact. The attainment of a sound method of procedure for determining the optimum design discharge and the most economical length of bridge (item 4) constitutes the ultimate goal in the present research program.

Application of Backwater to Length of Skew Crossing

A practical application to which the bridge backwater information may be used to advantage can be demonstrated by comparing the length and cost of skew bridges with the length and cost of equivalent normal crossings, on the basis of backwater. The procedure consists of choosing an existing normal stream crossing and computing the backwater which the bridge will produce for a given flood condition; then, holding stream conditions constant, compute the length of equivalent skew bridges which have the same effective waterway; i.e., produce the same backwater. This course of computation was followed for seven existing crossings and the results are shown in table 2. The normal length of these bridges varied from 75 to 2,000 feet and included both wingwall and spillthrough type abutments. The faces of the abutments under the bridge were oriented parallel with the flow, as shown in figure 4. This is the most efficient skew abutment shape. Abutments with faces at an angle to the flow require more length of bridge.

The ordinate in figure 4 is the ratio of skew length to normal length of crossing in percent, which is plotted with respect to the skew angle as abscissa and the contraction ratio M as a third variable. In the case of $M=1.0$ (no constriction of the stream), the skew length is simply $L_n/\cos\phi$. With constriction of the stream, the ratio L_s/L_n decreases with the value of M.

What is occurring can be better understood by referring to figure 5. The ordinate is the ratio of the projected skew length to the



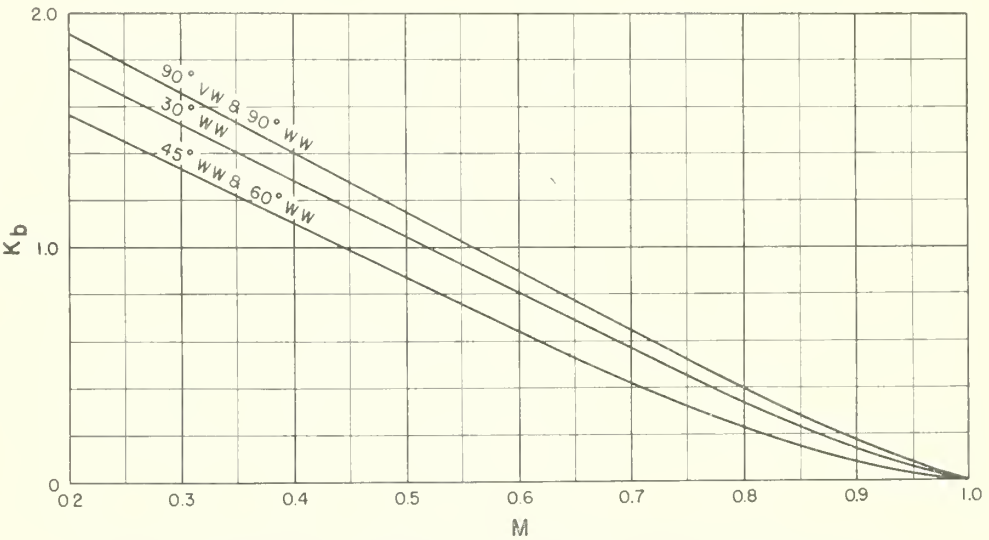


Figure 3.—Backwater coefficient K_b for wingwall abutments (base curve).

Table 1.—Comparison of computed Δh values with field measurements

Bridge number	Bridge length	Flood discharge	Average velocity under bridge	Contraction ratio, M	Computed backwater, h^*_1	Drop across embankments		Percentage difference, Δh
						Computed Δh	Measured Δh	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	Ft.	*C.f.s.*	*F.p.s.*		*Ft.*	*Ft.*	*Ft.*	
1	20	1,370	9.1	0.57	1.07	1.90	1.99	−4.5
2	84	4,340	6.8	.85	.21	.65	.70	−7.2
3	220	27,500	7.5	.90	.28	.76	.83	−8.5
4	83	5,240	8.6	.60	1.03	1.81	1.60	13
5	72	12,000	10.2	.83	.57	1.94	1.95	−.5
6	58	3,400	7.1	.82	.18	.61	.55	10.9
7a	44	2,620	7.8	.66	.63	1.23	1.15	6.9
7b	44	1,450	5.4	.70	.30	.66	.69	−4.4
8	112	9,640	9.0	.33	1.80	2.53	2.24	12.9
9	340	70,000	10.5	.90	.77	2.57	2.70	−5.0
10	68	7,230	(1)	(1)	----------	1.53	1.48	3.4
11	120	2,600	(1)	(1)	----------	1.70	1.61	5.6

1 Deck girder immersed.

Table 2.—Comparison of length and cost of skew bridges with normal crossings

Bridge identification	Skew angle, degrees	Contraction ratio, M	Backwater coefficients					Velocity head, $\frac{V_{n2}^2}{2g}$	Backwater, h^*_1	Projected bridge length	$\frac{L_s \cos \phi}{L_n}$	Length (skew)/ length (normal)	Cost (skew)/ cost (normal)
			Base k_b	Piers Δk_p	Eccentricity ΔK_e	Skew ΔK_s	Total K^*						
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
								Ft.	*Ft.*	*Ft.*			
A	0	0.90	0.12	0.09	0.07	0	0.28	1.70	0.76	340	1.0	1.0	1.0
	30	.90	.12	.09	.07	−.02	.26	1.84	.76	335	.98	1.14	1.22
	45	.90	.12	.09	.07	−.03	.25	1.89	.76	330	.97	1.37	1.54
B	0	.67	.48	.04	.15	0	.67	1.22	.89	2,000	1.0	1.0	1.0
	30	.665	.50	.05	.15	−.05	.65	1.26	.89	1,925	.96	1.12	1.18
	45	.66	.51	.06	.15	−.08	.64	1.30	.89	1,900	.95	1.34	1.50
C	0	.64	.55	.19	--------	0	.74	2.66	2.18	87	1.0	1.0	1.0
	30	.635	.55	.19	--------	−.06	.68	2.84	2.18	84	.96	1.12	1.22
	45	.63	.56	.19	--------	−.09	.66	2.90	2.18	80	.92	1.30	1.50
D	0	.62	.60	.03	.16	0	.79	1.65	1.41	1,100	1.0	1.0	1.0
	30	.62	.60	.04	.16	−.07	.73	1.79	1.41	1,025	.93	1.08	1.15
	45	.61	.62	.05	.16	−.11	.72	1.82	1.41	1,010	.92	1.30	1.45
E	0	.53	.92	.08	--------	0	1.00	1.11	1.19	630	1.0	1.0	1.0
	30	.52	.96	.09	--------	−.19	.86	1.27	1.19	600	.96	1.10	1.16
	45	.51	1.00	.12	--------	−.37	.75	1.46	1.19	575	.91	1.29	1.42
F	0	.46	1.13	.15	.04	0	1.32	.67	.93	1,075	1.0	1.0	1.0
	30	.43	1.16	.16	.04	−.26	1.10	.82	.93	990	.92	1.06	1.08
	45	.42	1.21	.19	.04	−.49	.95	.90	.93	925	.86	1.22	1.31
G	0	.46	1.06	.06	--------	0	1.12	.90	1.05	75	1.0	1.0	1.0
	30	.44	1.08	.08	--------	−.25	.91	1.09	1.05	69	.92	1.06	1.12
	45	.42	1.14	.09	--------	−.48	.75	1.35	1.05	64	.86	1.20	1.34

normal length while the other two parameters remain unchanged. For $M=1.0$ (no constriction), the ordinate is 1.0 for all angles of skew. With constriction of the stream, the projected skew length, required to produce the same amount of backwater, is shorter than the normal bridge length. This characteristic is to be expected, but the actual relationship has been until now entirely a matter of conjecture. The curves in figures 4 and 5 offer actual values which may prove useful in design.

A plot relating the cost ratio in percentage of skew to normal crossings for the same bridges is presented in figure 6. Again the parameters are the same except for the ordinate. The consistency is not of the same order found in the length curves since it was necessary to adjust span lengths and provide additional piers for the skew bridges. Also, the higher unit cost of skew construction and the increased length of embankments were considered. The cost was affected to a greater extent than the length by these factors. The criterion for determining span lengths for the skew crossings consisted of balancing the cost of superstructure against the cost of piers on an equal basis. The increase in cost of superstructures per square foot was assumed to be 5 percent for the 30° skew and 10 percent for the 45° skew.

For the purpose of comparison, the length varies from 107 to 115.5 percent of normal for the 30° skew, while the cost variation for the same range of contraction ratios is from 110 to 127 percent. In the case of the 45° skew, the length varies from 120 to 141 percent of normal compared with a cost variation of 130 to 158 percent for the same values of M.

Skew angles from 0° to 20° produce only a small increase in both length and cost over the normal crossing (figs. 4–6). As the angle increases above this value the curves get steeper and the length and cost rise more rapidly. In this same connection, it was observed in the course of the model studies that the hydraulic flow problems encountered with skew crossings for angles from 0° to 20° varied slightly from those for normal crossings. As the angle exceeds this value, the flow and scour problems increase in complexity.

Limitation of Backwater and Accommodation of Superfloods

Another application in which the backwater design information can be used is where approach roadways can be depressed to protect the bridge during floods of extreme proportions. Although it is seldom economically feasible to construct a bridge sufficiently long to accommodate the super type of flood, it is possible in many cases to design for a 35- or 50-year flood but make provision to pass flows of much greater magnitude with little or no damage to the bridge proper and, at the same time, keep the backwater within specified limits. The most effective way to present this is by actual illustration.

The stream at a proposed crossing has a low-water channel about 700 feet across, while in flood the stream may be a mile wide. Records show that within the past 50 years two floods approximating frequencies of 100 years have occurred on this stream, the last one destroying a bridge at the site. This is on a State route carrying a moderately heavy volume of traffic. A considerable amount of residential and business development occupying portions of the flood plain has sprung up within the last decade. It is therefore important from the traffic viewpoint that the bridge structure not fail or be out of service for an extended length of time during its expected life; and from the standpoint of life and property damage, it is desirable that the bridge backwater be limited to a definite figure for all flows. For the purpose of illustration the bridge is reconstructed to satisfy the above requirements and limit the backwater to 0.5 foot for any discharge likely to occur during the life of the bridge.

There is a choice here of designing a long bridge to take the full flow of the river for a 100-year frequency flood, keeping the embankments above high water at all times, or the

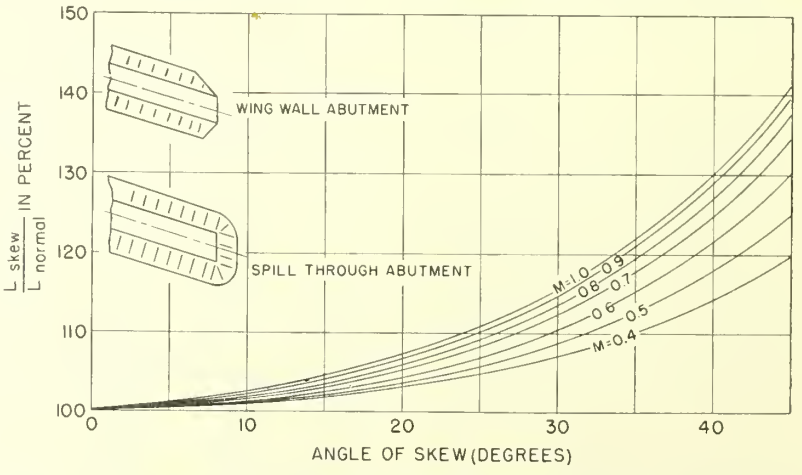


Figure 4.—Length ratio of skew to normal crossings for equivalent backwater.

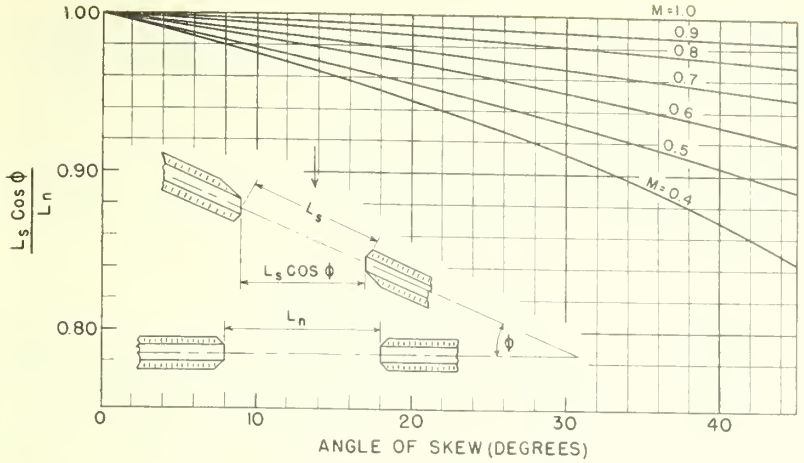


Figure 5.—Ratio of projected skew length to normal bridge length for equivalent backwater.

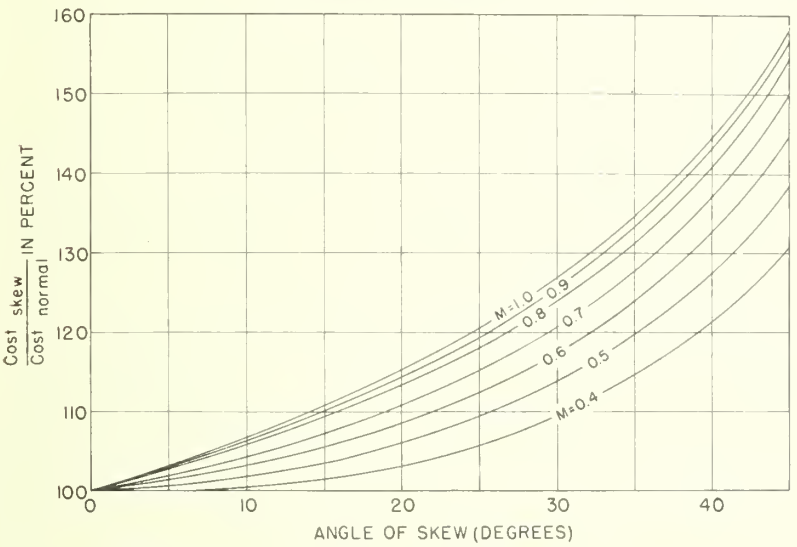


Figure 6.—Cost ratio of skew to normal crossings for equivalent backwater.

alternative of choosing a shorter bridge and using the one-half to three-fourth mile of roadway transversing the flood plain as a spillway during extreme high water. In either case the superstructure will be located above extreme high water at all times.

The case where the embankments are located above high water and the bridge is required to accommodate the entire flow will be investigated first. Figure 7 shows the backwater relative to length of bridge and discharge for this type of operation. In addition, scales have been superimposed showing flow recurrence interval at the top and bridge cost at the right. Were there no restriction on backwater, a bridge 1,500 feet long, producing 1.5 feet of backwater, might be a reasonable choice. But with backwater limited to 0.5 foot, it is observed that the bridge should be 2,250 feet long for a 50-year flood, or 2,600 feet long for a 100-year flood. From the scale on the right, the cost involved in reducing the backwater from 1.5 feet to 0.5 foot approximates $400,000 for the 100-year flood or 158 percent of the cost of the former. This comparison demonstrates how limitation of backwater can increase the initial cost.

How can limitation of backwater be accomplished with less cost? Let us now examine the alternative, the depressed roadway. Figure 8 demonstrates a very extreme case; the bridge has been shortened to 800 feet with approximately 3,500 feet of depressed roadway. The lower broken line, labeled "normal stage," represents the stage-discharge curve for the river prior to construction of the bridge. The upper broken line, labeled "stage without overflow," represents the stage discharge to be expected upstream from the 800-foot bridge without overflow. The difference between the two curves represents the backwater. For a discharge of 250,000 c.f.s. (50-year flood) the backwater is 2.5 feet, and for 300,000 c.f.s. (100-year frequency flood) the backwater approximates 4 feet. The limitation of 0.5 foot for backwater is reached at a discharge of 121,000 c.f.s. If the approach embankment is placed at elevation 87.5 so water will spill over the roadway for flows greater than 121,000 c.f.s., the backwater will decrease with further increase in discharge, falling off to about 0.1 foot for a discharge of 300,000 c.f.s. The backwater with overflow is represented by the difference between the lines labeled "normal stage" and "stage with overflow." As the roadway overflows, the discharge under the bridge increases slowly with upstream stage, while the roadway flow increases rapidly with stage. At stage 93.8 these lines intersect and the roadway is carrying as much flow as the bridge.

As the roadway is elevated, the backwater and flow characteristics remain similar to those shown in figure 8 but the bridge length must be increased if the backwater is to be limited to 0.5 foot for upstream stage level with the new roadway. Figure 9 demonstrates how the backwater varies with roadway elevation and length of bridge. The dashed lines denote the backwater which could be expected for several bridge lengths if flow over the road were not permitted. The solid lines demonstrate how flow over the roadway restricts the backwater to a maximum of 0.5 foot regardless of the discharge.

The depressed roadway not only serves to hold the backwater within limits but offers a means of accommodating the superflood without undue overloading of the bridge proper. It is true that the higher the approaches and the shorter the length of embankment, the longer the bridge must be for a given amount of backwater; nevertheless it is usually possible to set embankments for the 50-year flood stage and still retain this desirable safety valve feature.

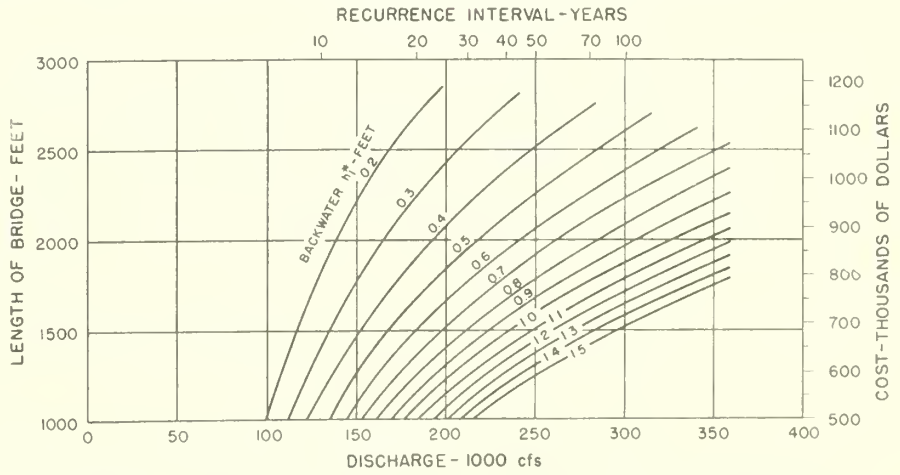


Figure 7.—Variation of backwater with length of bridge and discharge.

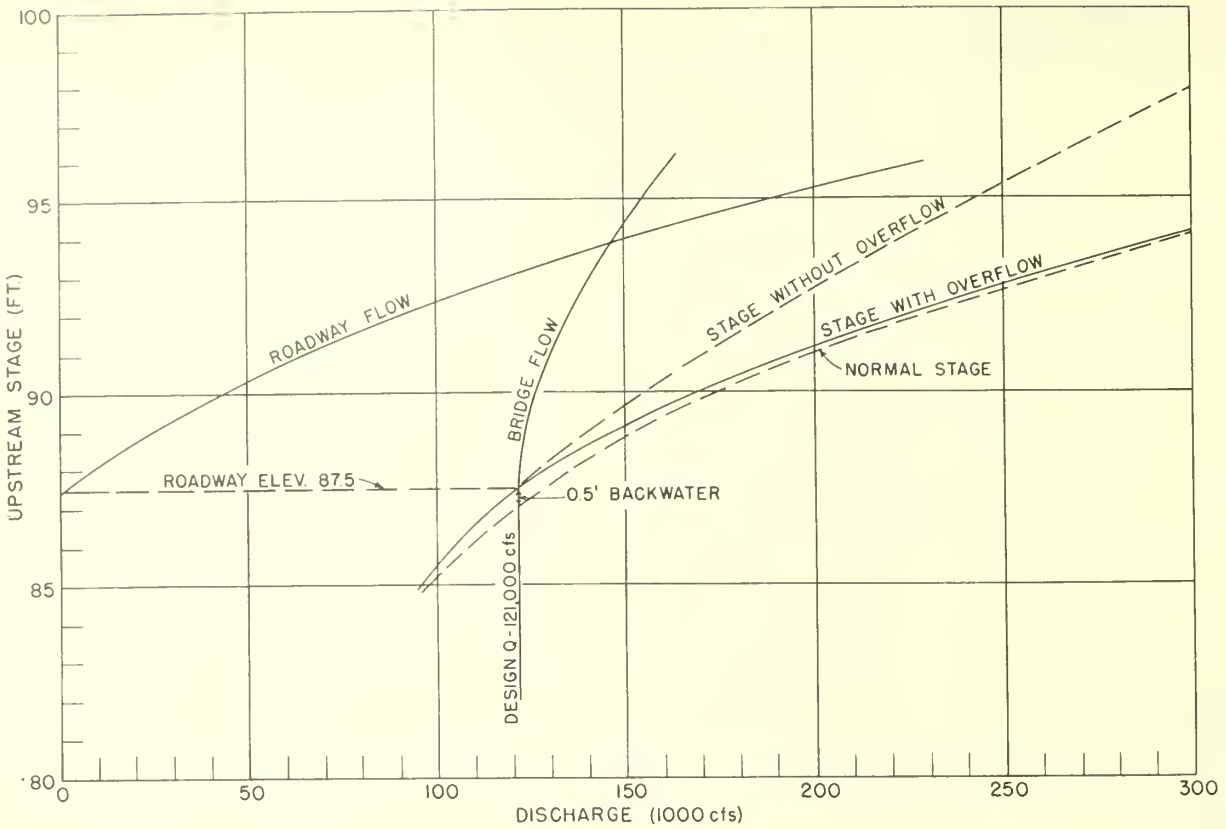


Figure 8.—Operation with depressed roadway and 800-foot bridge with backwater limited to 0.5 foot.

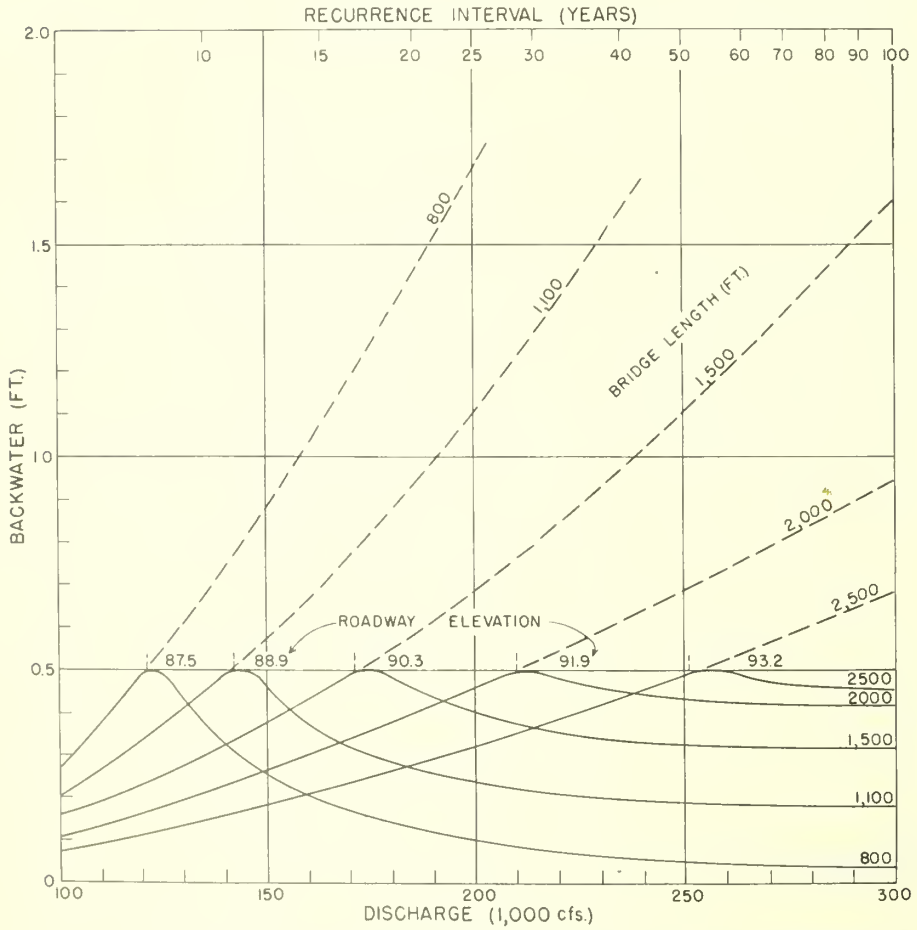


Figure 9.—Operation with depressed roadway for several lengths of bridge with backwater limited to 0.5 foot.

Current Research

The illustrations cited represent only a few ways in which the recently acquired bridge backwater information can be applied to waterway design. Gaps still remain which eventually will be plugged as reliable field data become available.

An equally important or second phase involving the hydraulics of waterways is the reasonable prediction of maximum scour depths at abutments and piers. Some information is already available for streams with alluvial beds (*3, 4*), and additional information will be forthcoming.

It will be found that the hydraulic analysis offers many variations of supposedly good waterway proportions. How then is an unbiased choice to be made? It is believed that this can best be accomplished through development of a generally acceptable type of economic analysis—which at the present does not exist—taking into account all tangible and certain intangible costs over the useful life of a bridge which may be incurred by highway agencies in building, operating, and maintaining a bridge and by highway users who travel over the bridge. In this way a fair monetary value can be assigned to each design, whereby comparisons can be made on a basis familiar to all parties concerned. Determination of the fundamental concepts on which such an economic analysis should be founded constitutes the third major phase of this research program now under consideration.

(Continued on page 231)

Assigning Traffic to a Highway Network

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by GLENN E. BROKKE, Highway Research Engineer

The construction of a highway network to serve a large urban area extends over a period of many years and involves the expenditure of several hundred million dollars. Before undertaking this long and expensive task, highway officials should know whether the network will adequately and efficiently accommodate future traffic.

This article describes the use of a high-speed computer in assigning estimated future traffic to a complete network of highways. In the process, traffic volumes are recorded on each individual highway link as well as the turning movements at each intersection. Thus, the engineer can investigate a series of alternate locations or design standards and measure their effect on the entire highway system.

The assignments can be made rapidly, inexpensively, and with a minimum amount of manual work. The program is also consistent and reproducible from one location to another. At the option of the user, provision can be made in the program to allow for one-way street operation, turn restrictions or delays, and peak hour traffic volumes by direction.

FOR many years highway engineers have been faced with the problem of estimating the volume of traffic that would use a proposed new facility. In addition, they would like to know how well this facility will continue to serve traffic in the future. A realistic solution to this problem is not simple. All zones in an entire metropolitan area have the potential of affecting any highway facility. The degree of this effect is governed by the location and design standards of the highway system as it now exists or as it may be subsequently improved. Thus, individual highway improvements can cause a realinement of traffic throughout the area.

Much research work has been done to provide a method for solving these problems. While the future will probably bring added refinements and even major changes in concepts, present knowledge is sufficient to warrant establishing at least an interim procedure for assigning present and future traffic to a network.

The necessity of handling a mass of information in a relatively simple but coherent manner is a characteristic of traffic studies. To process these data in a reasonable time, it has heretofore usually been necessary to use a series of short cuts, approximations, or judgment estimates, each exacting its toll in deviations from accuracy. It is now possible to employ electronic computers to provide a consistent solution, subject only to the limitation of knowledge of the behavior of traffic.

Utilization of Computers

The use of computers does involve the time-consuming operation of preparing a program. While it is desirable to allow flexibility in the program, this is not easily accomplished. Thus, to use an existing program, it is necessary to have access to the machine for which the program is written, to furnish input data in precisely the correct form, to willingly accept the logic that is incorporated in the program, and to accept the results in predetermined format.

The Bureau of Public Roads has written, tested, and used an IBM 705 program to predict the future distribution of trips.[1] The input required is the number of existing zone-to-zone trips and the growth factor for each zone. The program logic follows the Fratar formula. The output is the estimated number of future zone-to-zone trips. Either three different modes of travel or one mode to three different future periods can be processed simultaneously. For a city the size of Washington, D.C., with 500 zones, about one-half hour of IBM 705 time is required for each iteration. Two or three iterations are usually sufficient, thereby making the computer cost of obtaining future zone-to-zone trips somewhat less than $600 for a city the size of Washington.

Procedures for Assigning Traffic

With the present and future zone-to-zone trips available, the next problem is to estimate the loading of these trips on a highway network. To do this, some method of predicting the distribution of traffic between routes is required.

Three types of diversion curves are in current use: the time ratio curve developed by the Bureau of Public Roads (fig. 1), the distance ratio and speed ratio curve used in a Detroit study, and the time and distance differential curve used in California.

Time ratio curve

The Bureau's time ratio curve relates the percentage of trips using a freeway facility based on the ratio of the travel time via the freeway to the travel time via the best alternate route. The percentage of trips using the freeway varies as an S-shaped curve from 100 percent at a time ratio of 0.5 or less to zero percent at a time ratio of 1.5 or more. If the travel time via the freeway is equal to the travel time via the alternate route (time ratio=1.0), approximately 42 percent of the trips are assigned to the freeway.

Speed ratio curve

The speed ratio curves developed for the Detroit area transportation study consist of a family of curves where the percentage of freeway usage is related to speed and distance ratios. These curves are also S-shaped for normal conditions; and with a speed ratio of 1.0 and a distance ratio of 1.0 approximately 45 percent of the trips are assigned to the freeway. Since these curves represent a three-dimensional surface with an undefined mathematical relationship, they are difficult to use in a computer application.

Time and distance curve

The California time and distance curve consists of a family of hyperbolas; and with equal time and distance on the freeway as compared with the best alternate route, 50 percent of the trips are assigned to the freeway. These curves can be expressed in the equation,

$$P=50+50\ (d+\tfrac{1}{2}t)\ [(d-\tfrac{1}{2}t)^2+4.5]^{-\frac{1}{2}}$$

Where:

P=Percentage of freeway usage.
d=Distance saved in miles via the freeway.
t=Time saved in minutes via the freeway.

Problems Encountered in Assigning Traffic

The development of an assignment procedure has two major difficulties: the measurement of the minimum traveltime between each pair of zones over the arterial network and then over the entire highway network including the contemplated freeways, and the accumulation of the assigned volumes on the various segments of the highway system.

The Washington, D.C., Regional Highway Planning Committee, having recently completed an origin-destination study and having predicted the zone-to-zone movements to 1980, wished to assign this traffic to a highway network using the Bureau's time-ratio diversion curve. The effort involved in accomplishing this task for an area the size of Washington was clearly beyond the range of

[1] *Evaluating trip forecasting methods with an electronic computer*, by Glenn E. Brokke and William L. Mertz. PUBLIC ROADS, vol. 30, No. 4, Oct. 1958, pp. 77–87.

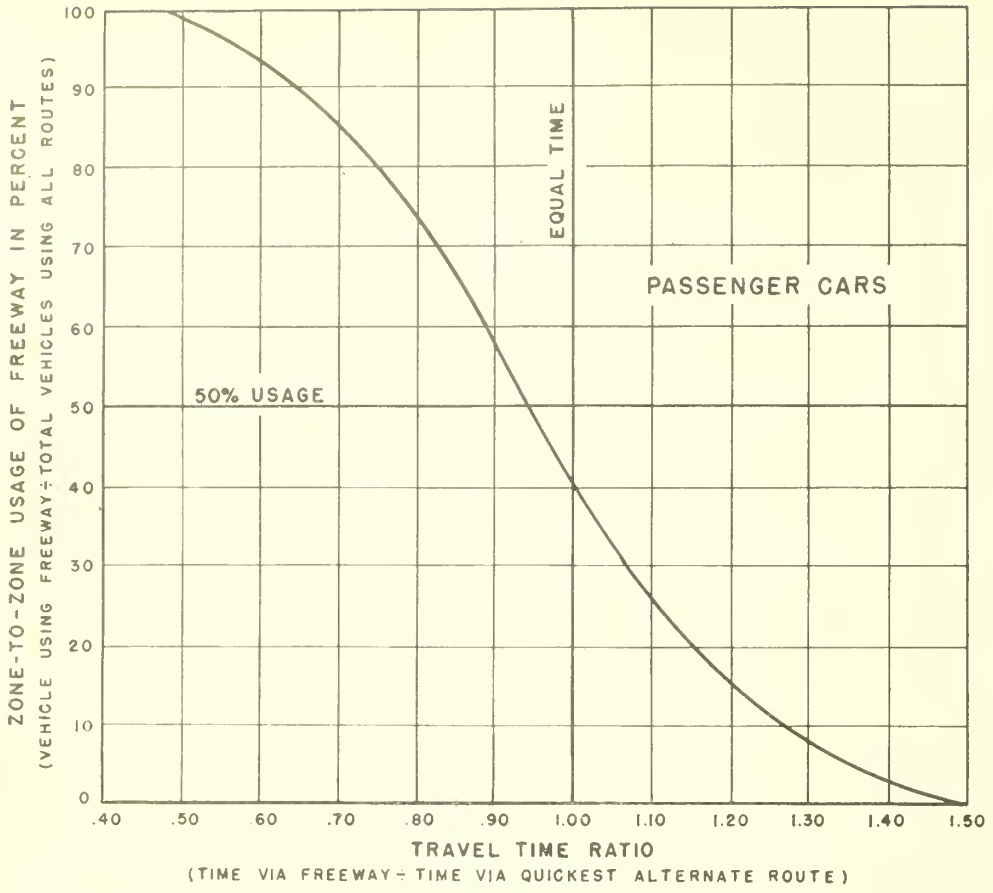


Figure 1.—Time-ratio diversion curve.

practicability, unless an electronic computer could be used for a major portion of the work. The Bureau of Public Roads offered technical assistance. It was soon discovered that no suitable program was available which would handle the complexities of Washington traffic movements. Hence, it became necessary to develop a Washington assignment program.

Development of Minimum Path Principle

At about this time Dr. J. D. Carroll, Jr., director of the Chicago Area Transportation Study, and his staff discovered a method of determining the minimum path through a highway network. Also Dr. A. J. Mayer, director of the Detroit Area Traffic Study, and his staff carried this minimum path principle along somewhat different lines toward an assignment procedure. Both organizations were visited and each was entirely cooperative and responsive in outlining their procedures and ideas on the problem.

Since all diversion curves being considered are based on the relationship between the traveltime (and distance) on the most favorable freeway route and the traveltime (and distance) on the most favorable alternate route, the initial problem is to determine which of the freeway routes and which of the alternate routes are truly the most favorable.

The difficulty of this problem can be appreciated most easily if a rectangular street network is considered. To arrive at a point 4 blocks east and 4 blocks south of an origin, there are over 40 different routes or paths that appear approximately equal to the eye as far as traveltime (or distance) is concerned. However, by accurately adding the time (or distance) values on each of the segments involved for each route, the route with the least overall traveltime can be selected. This selected route is the minimum traveltime route or minimum path.

It is true and probably apparent that the longer the trip, the more alternate routes there are available between two points. For travel across an entire city, there may be literally thousands of alternate paths or routes and the initial problem of determining which path is the minimum appears rather difficult.

Fortunately, Dr. Carroll and his staff were able to use a procedure developed by Edward F. Moore[2] of Bell Telephone Laboratories, Inc. The same basic method is used in this program and essentially consists of accumulating the minimum time and path from a central point to an ever-increasing circle of points surrounding this central point.

Advantages of Minimum Path Principle

The value of the minimum path principle can hardly be overestimated in the solution of the assignment problem. The distance and traveltime on each segment of the highway network are determined and fed into the computer. These initial time and distance measurements are required for any of the methods being used, and are neither easier nor more difficult to obtain for the minimum path procedure. Once the traveltimes are in the computer, however, very substantial advantages begin to accrue.

The most obvious advantage is the savings in man-hours. On the optimistic basis that the best route of travel between a pair of zones can be located and measured in 3 minutes by manual methods, approximately 2 man-years of labor would be required to find the traveltime via the arterial network and via the freeway network for the 40,000 zone-to-zone volumes occurring in the Washington area. The computer can absorb all of this manual work at the rate of about 2 computer-hours being equivalent to 1 man-year of manual computation.

A second advantage is the increase in accuracy and consistency. A manual determination of the best route has two sources of frequent errors. The routing selected as the minimum path may actually be longer than some other path. Secondly, an error may be made in adding the time intervals that make up the selected path. The minimum path program, however, tests all possible paths, selects the minimum, and adds the time intervals unerringly.

A third advantage, which is somewhat more obscure, is the ability of the computer to take additional factors into account. For example, the computer can be rather easily instructed to test a routing and insert a turn penalty whenever a right- or left-hand turn occurs. To add this or a similar complication into a manual procedure would be entirely impractical.

Washington Traffic Assignment Program

The Washington traffic assignment program is in reality a library of programs that can be selected in any desired order through the use of a master control program. To use this library of programs certain conventions must be observed, and to understand these conventions a few definitions are required.

Definitions

Node.—A node is any specific point on the highway system that is needed for identification purposes. Primarily, nodes are used to designate zone centroids and highway intersections. They are identified by number.

Link.—The portion of the highway system between two nodes is a link. To avoid needless complication only the "through" or more important highways are identified by actual location. Links are identified by the two node numbers which terminate the link.

Route.—A group of connecting links between a pair of zones is the route of travel between these zones. If a particular route has a shorter traveltime than any other

[2] *The shortest path through a maze,* by Edward F. Moore. Paper presented at the International Symposium on the Theory of Switching, Harvard University, 1957.

route, it is called the minimum time path. If distance instead of time were the criterion, a minimum distance path could be similarly described.

Tree.—All minimum path routes from one particular node to all other nodes in the system constitute the tree for that particular node. In practice, trees need to be built only for the zone centroid nodes.

Program conventions and limitations

1. No more than four links may meet at any node. To accommodate five or more links which would otherwise intersect at a single node, it is necessary to separate the one node into two (or more) nodes with zero time and distance between them.

2. No node may be numbered more than 4,000.

3. The node numbers must be arranged in sequence in four separate groups in the following order: *group A*, zone centroids starting with number 1; *group B*, four-way arterial nodes; *group C*, two- or three-way arterial nodes; and *group D*, freeway nodes.

4. From each four-way node there must be at least one link to a numerically larger node number.

5. The zone-to-zone trip cards must be in sort by the first zone before being placed on tape.

6. To be able to insert a turn penalty for right and left turns, it is necessary to designate each link as positive or negative. Movements between links of the same sign involve no turn. To accommodate diagonal or curving streets, a flag position is also available to change signs as needed.

The Washington traffic assignment program library consists of the following individual programs:

0—Master control.
1—Build trees.
2—Load arterial network (all or nothing).
3—Load entire network (time ratio curve).
4—Sum vehicle-miles and vehicle-hours.
5—Convert link data from decimal to binary.
6—Make freeway corrections to link data.
7—Convert trip volumes from decimal to binary.
8—Correct trip data.
9—Prepare time-ratio diversion table.

These programs are on magnetic tape and may be used in any desired order through the master control program.

Input Data Requirements

Zone-to-zone trips

Each of the zone-to-zone trip volumes must be represented by a trip card identifying the two zones (or stations) involved and the number of trips between them. Zero volumes need not be represented by a card except that each zone other than the last one must be represented by at least one card (zero volume if necessary) when arranged in sort by the first identifying zone number. If not already accomplished in a previous stage, the zones and stations must be renumbered to form an unbroken sequence starting with number 1. When placed on tape, the zone-to-zone cards must be in sort by the first identifying zone number.

Highway link data

The highway network must be described properly for computer application. This is accomplished by listing each link of the highway network on a coding sheet. The listing consists of the two identifying nodes together with the sign, a flag if needed, the traveltime in minutes and hundredths, and the distance in miles and hundredths. These data are then key punched with one link to a card. The cards are then duplicated, reversing the two identifying nodes and adjusting the sign and flag, so that in the final deck each link is actually listed on two cards. The data on the cards are then transferred to magnetic tape.

Program Operation

Program 0, master control

This control sets the parameters for the specific area in which traffic is being assigned and permits the choice of any of the subroutines included in the program library. As input, it requires the number of zone centroids, the number of four-way intersections, the number of two- or three-way intersections, the number of freeway intersections, and the amount of turn penalty in minutes and hundredths.

In addition, it has been necessary to scale the time and distance values into units which will economically use the memory availability of the computer. Therefore, the maximum traveltime on any link plus the turn penalty is equated to 63. The time values on all other links are converted to sixty-thirds of this maximum traveltime link. For the same reason the maximum distance link is likewise equated to 63, and all distances converted to sixty-thirds of this maximum distance link. It should be noted that the maximum traveltime link and the maximum distance link need not be the same link.

The maximum traveltime link and the maximum distance link are also necessary inputs to the master control program. All of the other programs are then set with the specific characteristics entered through this master control program.

Program 5, convert link data from decimal to binary

In program 5 the computer edits the link data for impossible codes, scales the time and distance values to appropriate units, converts all data from decimal to binary, and packs the information to fit exactly into a block of computer memory. The output is a binary coded tape containing this large block of information.

Program 6, make freeway corrections to link data

The most difficult problem in determining the freeway route is to arrive at some method which will compute a minimum freeway path even though it is longer than an arterial street path. This is necessary because some diversion to a freeway exists even if the time ratio is more than 1.0. To retain the advantage of the minimum path method and still obtain a freeway time longer than arterial time, it was decided to temporarily halve the time on the freeway links. Once the tree has been established, the time values are corrected.

In program 6 the previous arterial links are modified as needed by the addition of the freeway nodes; the freeway links are inserted into the system with their time values cut in half; and the information is converted to binary and packed into a block of memory. The memory is then written out on tape in binary code.

Program 7, convert trip volumes from decimal to binary

The tape containing the zone-to-zone trip volumes is edited in program 7. The numbers are converted from decimal into binary, and all of the trips from the first-listed (or origin) zone to all other zones are packed into one record block which is written out on tape. There must be a trip record block for each zone except the last (highest numbered) zone.

Program 8, correct trip data

If in the process of editing or through subsequent checking it is found that some of the zone-to-zone trips are in error, the values may be corrected in program 8 without rerunning the entire program.

Program 9, prepare time-ratio diversion table

Program 9 builds the diversion curve table for converting time ratio to percentage diversion. At the present time the traffic diversion curve plotted in figure 1 is incorporated in the program.

Program 1, build trees

Program 1 determines the minimum path from each zone to all nodes in the highway network. If only the arterial links are used, the program builds arterial trees. If the freeway links are also included, the program builds freeway trees. Thus, the previously prepared link data are the input for this program.

The program instructs the computer to set aside a block of memory for the tree with each memory word of the block corresponding to an actual node on the highway system. The memory words in the block are in the same sequence as the node numbers. Thus the position of the memory word identifies the node number.

In addition, each word in the tree memory block will contain two major items of information: (1) the preceding node through which the route has passed in building the tree, which is called the back node, and (2) the total elapsed time from the tree centroid to the node represented by this memory word.

Each memory word is initially set to the largest possible value. The computer then starts building the tree from zone centroid 1 in the following manner:

A. Since the tree is being built from node 1, the first step is to set the back node and the

elapsed time in memory word No. 1 to zero. At the same time this node is listed in an elapsed time sequence table in the zero time slot.

B. The computer then takes the minimum entry in the elapsed time table, erases this entry, and from the link memory block, finds all links that emanate from this node, which can be called node A.

C. At the end of each of these links there is a second node which can be called node B. The link time from node A to node B, plus a turn penalty if required, is added to the total elapsed time at node A to give the total elapsed time from the tree centroid to node B. The machine compares the computed elapsed time at node B with the previously established elapsed time stored in the word represented by node B in the tree memory. If the new time is less than the stored time, it replaces the stored time and node A replaces the previously stored back node. At the same time node B is stored in the elapsed time sequence table in the appropriate time slot. If, however, the new time is equal to or greater than the previously stored elapsed time, the route is not a minimum path and the computer discards this value.

D. When all of the links emanating from node A have been completed in this manner, the computer again selects the minimum entry in the elapsed time sequence table and repeats the process.

E. When all values in the elapsed time sequence table have been used, the tree from zone centroid No. 1 has been completed and the tree memory is written out on tape.

F. The computer then proceeds to zone 2 and builds the tree from this zone in exactly the same manner.

G. When trees have been built from all zone centroids, the arterial tree routine has been completed. For the Washington, D.C., area, about 450 trees are needed.

The program has been written so that any freeway pattern may be superimposed on the arterial street network without destroying the arterial trees already developed. The freeway trees are built in exactly the same manner as the arterial trees, except that the freeway links as well as the arterial links are included in the input data.

Program 3, load entire network (time ratio curve)

The previously completed freeway and arterial tree tapes and the zone-to-zone trip tape become input for program 3. In addition, the relationship between time ratio and percentage diversion is placed in the computer memory. The program then performs the following operations:

A. The arterial tree for node 1 (also zone centroid 1) is read into a block of memory.

B. The freeway tree for node 1 is read into a separate block of memory.

C. The zone-to-zone trips from zone 1 are read into a third block of memory.

D. The trips between the first pair of zones initiate the following action: (1) The destination zone of the trip becomes the first entry in the arterial route. (2) From the arterial tree, the back node of the destination zone becomes the second entry in the arterial route. (3) The back node of the second entry becomes the third entry for the route and so on until the route reaches zone centroid 1. (4) The freeway route is established in the same manner with the corrections to travel-time on the freeway links being made. (5) The arterial route is compared with the freeway route. (*a*) If the routes are identical, all of the trips are accumulated on the arterial routing in a block of memory where each word represents a corresponding highway link. (*b*) If the routes are different, the two points of choice are determined. The travel-time via the freeway and via the arterial system between points of choice is computed and converted to time ratio and then to percentage diversion. The freeway traffic is accumulated in memory via the freeway route, and the arterial traffic is accumulated in memory via the arterial route. (6) At all four-way intersections, two of the turning movements are recorded separately in a turn table so that in the final analysis all turning movements are available.

E. The remaining trips from zone 1 are handled in the same manner after which the trees and trips from zone 2 replace those of zone 1 and the process is repeated.

F. This procedure is continued until all zone-to-zone trips have been read by the computer, at which time the accumulated volumes are written on tape in decimal form. The decimal tape is printed on peripheral equipment. The printed output is the traffic load on all segments of the entire network including all turning movements.

Table 1.—Computer time required to run through the various programs in the library

Program	Units: Nodes	Units: Zone-to-zone cards	Total computer time	Rate
			Sec.	
Convert links to binary	543	-----	25	¾ minute per 1,000 nodes.
Convert volumes to binary	---	3,488	60	3 minutes per 10,000 cards.
Build trees (102 zones)	543	-----	280	8½ minutes/100 zones/1,000 nodes.
Load network (time ratio)	---	3,488	275	13 minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Load network (all or nothing)	---	3,488	97	4⅔ minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Compute vehicle-miles and vehicle hours [1]	543	-----	35	1 minute per 1,000 nodes.

[1] Includes conversion of link volumes and turning movements from binary back to decimal, written out on tape in an appropriate format.

Program 2, load arterial network (all or nothing)

The library also includes a program for loading all trips on the shortest route. This is accomplished by reading only the arterial trees into memory and loading all trips on the routing established by these trees.

Program 4, sum vehicle-miles and vehicle-hours

The vehicle-miles and vehicle-hours on the freeway network, on the arterial system, and on the local system are then computed and printed.

Computer running time

The entire program library has been completed and a major portion tested in the Virginia portion of the Washington metropolitan area. This area consists of 102 zones, 543 nodes, and 3,488 zone-to-zone movements. The time required to run through the various programs in the library is shown in table 1.

Using the rates given in table 1, it is possible to estimate the computer time required for one complete assignment for any area. For example, it is expected that the Washington, D.C., metropolitan area will consist of about 450 zones, 3,100 nodes, and 40,000 zone-to-zone trip cards. The estimated computer time for this area is shown in table 2. An

Table 2.—Estimated computer time required for 1 complete assignment of traffic in the Washington, D.C., metropolitan area

Program	Units: Nodes	Units: Zone-to-zone cards	Total computer time	Rate
Convert arterial links to binary	3,100	---	3 min	¾ minute per 1,000 nodes.
Convert freeway links to biuary	3,100	----	3 min	¾ minute per 1,000 nodes.
Convert volumes to binary	---	40,000	12 min	3 minutes per 10,000 cards.
Build arterial trees (450 zones)	3,100	----	120 min	8½ minutes/100 zones/1,000 nodes.
Build freeway trees (450 zones)	3,100	----	120 min	8½ minutes/100 zones/1,000 nodes.
Load network (time ratio)	---	40,000	90 min	13 minutes per 10,000 cards $\times\left[\frac{\text{Number of nodes}}{500}\right]^{\frac{1}{4}}$
Compute vehicle-miles and vehicle-hours.	3,100		3 min	1 minute per 1,000 nodes.
Total computer time	-	-	5 hr. 51 min.	
Set up and control time	-	-	10 min.	
Contingency time (10 percent)	-	-	34 min.	
Grand total computer time	-	-	6 hr. 35 min.	

IBM 704 computer with 32,000 words of memory will cost from $350 to $400 per hour. Thus the machine cost for one assignment for the Washington area will range from $2,200 to $2,600.

Characteristics of Traffic Assignment Program

The accuracy of the assignment program rests basically upon the accuracy of the assumption that traffic divides between routes in accordance with the time-ratio diversion curve. In any particular city, the accuracy of this assumption can be checked by assigning present trips to the existing highway system and checking against current traffic counts. If better criteria are subsequently developed which improve the distribution of traffic among routes, they will be incorporated into the program.

The program as written has a considerable amount of flexibility. Changes in the extent or location of the proposed freeways can be tested by merely altering the freeway network and rerunning the program. If any of the proposed highway segments are loaded beyond capacity, the traveltimes on these sections can be adjusted and the program rerun until there is a balance between capacity and traveltime on each segment. If directional zone-to-zone trips are available, the program can give directional assignments. Thus, traffic on one-way streets or ramps can be directly computed.

It is likely that the future use of the program will develop additional subroutines which will be useful in designing highways. By way of illustration, consider the case of a ramp connection immediately before an interchange between freeways. If the predominant flow of traffic from the ramp turns left at the freeway interchange, it may be advantageous to bring the ramp into the freeway from the left to avoid the confusion of weaving this traffic across the freeway. By suitable instructions, the computer can develop these or similar data.

The program is written for an IBM 704 computer with 32,000 words of memory. If there are less than 900 nodes in the highway system, an 8,000-word memory will be sufficient. In addition to memory capacity, it is essential that the computer have extremely fast access time to all memory positions. This consideration, at least for the present, precludes the use of computers which rely on a magnetic drum memory.

Acknowledgment

The development of this traffic assignment program represents the efforts of a number of specialists in the field. Besides Dr. Carroll and Dr. Mayer, who are mentioned earlier in the article, valuable contributions were made by the staff of the General Electric Computer Laboratory, Tempe, Ariz.; Mr. W. F. Boardman, Acting Technical Director, Regional Highway Planning Committee, Washington, D.C.; and Mr. W. L. Mertz, Bureau of Public Roads.

Use of Backwater in Bridge Design

(*Continued from page 226*)

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(3) *Scour Around Bridge Piers and Abutments*, by Emmett M. Laursen and Arthur Toch. Bulletin No. 4, Iowa Highway Research Board, Iowa State Highway Commission, Des Moines, May 1956, 60 pp.

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Vehicle Acceptance Rates of Parking Areas

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by ARTHUR A. CARTER, Jr., Highway Research Engineer, and BESSIE A. HAYES, Statistical Assistant

VEHICLE acceptance rates of parking facilities, or more specifically, the number of entering vehicles that can be accommodated in a given period of time by a single entrance, are of primary importance in parking lot operation. In recent years there has been considerable research on this subject in downtown areas, including outdoor parking lots, and parking structures of various types, as well as studies of curb parking usage. Consequently, curb parking habits and the vehicle acceptance rates of city parking lots and structures are quite well known.

Considerably less research effort, however, has been devoted to vehicle acceptance rates at parking facilities in suburban and rural areas. In these outlying areas parking facilities are quite large and, except for those at suburban shopping centers, are used primarily for long-term parking. They include parking areas at industrial plants, racetracks, military installations, stadiums, and similar places. Generally, there is little turnover of vehicles at these parking areas. Instead, a concentrated influx of vehicles occurs over a relatively short period, with a similar discharge surge at a later time. Few detailed studies have been made of the traffic flows into such parking areas, although many overall counts have been made of the total number of vehicles accommodated during the heavy influx period.

Early in 1958, as one of its responsibilities in connection with civil defense highway planning, the Bureau of Public Roads was requested to determine the vehicle acceptance rates of large parking areas comparable to those which might be provided at refugee reception centers. This presented an ideal opportunity for the Bureau to accomplish two purposes with one study. The results would fill the existing void in available information regarding vehicle acceptance rates of outlying parking areas, and provide the information needed for emergency planning. For the first purpose, field studies of the operation of large improved parking areas were essential. Secondly, operations at these improved lots could be accepted as comparable to the best operation expected at refugee centers, since most of these centers would have relatively unimproved parking areas such as large open fields.

Accordingly, after Bureau of Public Roads personnel had made pilot studies near Washington, D.C., the cooperation of the State highway departments was enlisted in conducting studies at major parking areas throughout the country. The response was excellent, with 24 States supplying data obtained at a total of 74 parking areas. Included were studies at 125 entrances, of which 71 were found to include capacity conditions suitable for analysis.

Scope of Study

The data received were classified according to five types of entrances, depending upon the driving maneuver involved in the approach to the parking lot. The types were as follows: (1) straight approach (no turning movement); (2) 90° right turn; (3) 90° left turn; (4) oblique angle turn, right; and (5) oblique angle turn, left. Within each category, the lot entrances were further identified either as familiar to the drivers using them, such as those at industrial plants, or as unfamiliar lot entrances, which were infrequently visited by individuals, such as those provided for various special events.

Except for a few special reports, the analyses were based entirely upon the determination of approach volumes per hour per lane which actually entered the parking facility. Where over 50 nonentering vehicles per hour were intermingled in an approach traffic flow, or where opposing traffic was significant, the study was rejected. From each individual lane study, the peak-hour and the peak 10-minute volumes were recorded, and the peak 10-minute rate was expanded to a full-hour rate.

Observations made during the pilot study, as well as comments made by several of the observers in the field, indicate that an acceptance rate attainable for a full 10 minutes can be maintained for a full hour, provided that adequate maneuvering and parking space remains available within the lot and that a continuing capacity flow of vehicles arrives. This presumably would be the situation at a refugee reception center, although at the facilities studied, peak arrival rates seldom lasted for more than a half hour. Results are therefore given in terms of vehicles per hour per lane, as obtained by expanding the peak 10-minute rates (table 1). They are average values considered to be typical for the conditions shown.

Results of Study

Straight approach entrances

In the 11 studies involving unfamiliar, straight approach entrances, vehicle acceptance rates ranged from 770 to 940 vehicles per hour per lane. The average rate was 850 vehicles per hour per lane. Most drivers at this type of entrance, as well as most other unfamiliar entrances, came to a near stop in their approach.

Studies at familiar, straight approach entrances showed an average acceptance rate of 1,100 vehicles per hour. However, extremes to this average were reported. A rate of only 600 vehicles per hour was found at one industrial plant parking location. At another plant location, where an estimated speed of 20 m.p.h. was maintained through the entrance and vehicles could spread out into several lanes beyond the entrance, a rate of 1,900 vehicles per hour was recorded. This rate is somewhat higher than the predicted possible capacity of a single traffic lane at 20 m.p.h. The study, however, was made at an unusually well-operated industrial plant parking lot where roadway speed could be maintained through the gates and where parking had become a daily routine to all drivers.

90° turns

The average entrance rate at unfamiliar parking areas involving a 90° right turn was about 750 vehicles per hour. A one-third

Table 1.—Vehicle acceptance rates of large parking areas in 24 States

Approach to entrance	Number of studies	Average acceptauce rates—vehicles per hour per lane	
		Unfamiliar entrance [1]	Familiar entrance [2]
Straight approach (no turning movement)	20	850	1,100
90° right turn	15	750	1,000
90° left turn	24	830	900
Oblique angle, right	8	650	1,000
Oblique angle, left	4	720	[3]

[1] Includes racetracks, stadiums, and other facilities uot frequently visited by the same individuals.
[2] Includes industrial plants, military bases, and other facilities where the same drivers euter daily.
[3] No data available.

increase in the flow, to 1,000 vehicles per hour, resulted where drivers were familiar with the parking location.

At entrances involving 90° left turns, the results were surprisingly high in several studies at both familiar and unfamiliar locations. While volumes at several entrances were in the 600–700-vehicle-per-hour range, observers at other parking areas reported rates from 1,100 to 1,300 vehicles per hour, even though speeds were estimated as 10 m.p.h. or lower. One study location was particularly comparable to refugee center conditions. At this location, where vehicles from one lane rather promiscuously crossed a shallow ditch at various points to enter a grass lot, a rate of 850 vehicles per hour was attained. This was approximately equal to the average of 830 vehicles per hour for all of the unfamiliar lots, which indicates that unimproved lots can operate just as well as improved lots. The rate at familiar entrances showed only slightly better operation, averaging 900 vehicles per hour.

The results of the study show that the average left-turn rate was somewhat higher than the average right-turn rate at unfamiliar lots. This is contrary to the usual finding in intersection capacity studies that left turns involve more delay than do right turns. At the average intersection, drivers in making left turns are often confronted with opposing through traffic. In the parking studies, drivers were not faced with opposing traffic in negotiating left turns. Under these circumstances, left turning movements might be made from either lane of a two-lane highway, depending upon whether the approach road was operating one-way or two-way with practically no opposing traffic. When left turns are made from the left lane, the turn is just as sharp as the usual right turn, and thus the acceptance rates for either right or left turns should be similar. When left turns are made from the normal right-hand approach lane, a wider turn is involved which can be made much more easily. This very likely explains the more favorable rate for 90° left turns reported in table 1 for unfamiliar entrances as compared with 90° right turns.

Oblique angle entrances

Limited data obtained for oblique angle entrances indicated that left-turn rates again were somewhat higher than those for right turns. At unfamiliar locations, left turns averaged 720 vehicles per hour, while right turns averaged only 650 vehicles. At familiar locations, only data for right oblique entrances were available. These showed an average flow of 1,000 vehicles per hour. It can be assumed that left oblique entrances would approximately equal this capacity.

Effect of Gatemen on Traffic Flow

When these studies were being planned, it was assumed that collection of fees or inspection of passes would have an adverse effect on the traffic flow. The results, however, showed no consistent difference between lots where fees were collected or passes checked (applicable to about one-third of the parking areas studied) and those where access was unrestricted. At the average lot, entrance speeds were very low regardless of whether or not gatemen were present. Similarly, at these low speeds, it was found that lane width was not significant. All approach and entrance lanes studied were at least 10 feet wide, so it is possible that narrower rural roads might yield lower rates.

Practical Acceptance Rate

As a result of these studies, it appears that 800 vehicles per hour per entrance lane could be considered the practical maximum vehicle acceptance rate for large unfamiliar parking areas, regardless of the turning movement involved. This rate is possible if adequate parking areas exist to absorb all entering vehicles, if attendants prevent congestion within the lot, if the feeder roadway has a continuous backlog of vehicles, and if disabled vehicles can be rapidly cleared from the highway. It should be emphasized that dry weather conditions must prevail to accommodate 800 vehicles per hour per lane where unpaved entrances and parking areas are involved. Rain or snow could easily make such areas totally unusable.

If all drivers are familiar with an entrance, a rate of at least 1,000 vehicles per hour, and frequently more, may be possible. In those unusual situations where well designed straight-through approaches permit traffic to maintain full highway speed until distribution to specific sections within the lot begins, it may be possible to attain the maximum possible capacity of a moving traffic lane, 2,000 vehicles per hour. Certainly, however, this figure should not be used in planning.

It should be noted that studies were made only of traffic entering the various facilities before events, since this was the movement considered applicable to civil defense planning. Only limited data were received regarding exit flow from these same facilities, but it was evident that exit rates were somewhat higher than approach rates.

New Publication

The *Catalog of Highway Bridge Plans* has been assembled by the Bureau of Public Roads primarily for use by highway departments to facilitate the exchange of designs on a national basis. The program for the exchange of bridge designs was suggested by many State highway departments and by committees and members of the American Association of State Highway Officials.

The States were requested to submit their available bridge designs dating from 1950 for H20–S16 loading or heavier. The publication is divided into 13 sections and lists in tabular form over 4,500 designs. The data are assembled by States according to span types, simple, continuous, or cantilever, in their relevant sections and cover rolled beam spans with and without composite action, steel deck girder spans riveted or welded, with and without composite action, steel truss spans, reinforced concrete slab, deck girder, box girder, and rigid frame spans, prestressed concrete slab, girder and box girder spans, steel arch spans, reinforced concrete arch spans, movable bridges of lift span and bascule span types, suspension bridges, pedestrian overcrossing structures, traffic sign structures, and tunnels.

The last section (XIII) contains quantity charts which may prove useful in making design comparisons. These charts show structural steel quantities for rolled beam, deck plate girder and truss spans, and concrete quantities for reinforced concrete spans. Insufficient data were available to prepare charts covering reinforcing steel.

The *Catalog of Highway Bridge Plans*, a 167-page publication, may be purchased from the U.S. Government Printing Office, Washington 25, D.C., at $1 per copy.

Traffic Signals and Accidents in Michigan

BY THE TRAFFIC OPERATIONS DIVISION
BUREAU OF PUBLIC ROADS

Reported[1] by DAVID SOLOMON,
Highway Research Engineer

TO the general public, the installation of a traffic signal often implies that accidents will be substantially reduced. Studies have frequently shown, however, that this is not always true. One such study was recently conducted in Michigan. From 1946 to 1957 the Michigan State Highway Department made a detailed accident study at each of 89 intersections where traffic signals were installed. The intersections, distributed throughout the southern half of the State, were located in and near cities as well as on rural highways. Data were collected at each intersection for 1 or 2 years prior to installation of the signals and for an equivalent period afterwards.

The types of intersections included in the study ranged from simple three-leg to five- and six-leg and other complex intersections. As might be expected, more than half of the intersections had four approach legs without medians or islands to separate opposing streams of traffic. The four-leg divided intersections were, in most cases, divided on the main highway only. Of the 89 intersections studied, 39 were signalized with standard stop-and-go controls and 50 with flashing beacons. These beacons flashed yellow on the main street or major highway, and red on the side street or minor highway.

At 38 of the 89 intersections, traffic data were available. The average daily traffic volumes entering from all approaches at these intersections averaged 20,200 vehicles a day for stop-and-go signals and 8,000 vehicles a day for flashing beacons.

[1] Acknowledgment is made to the Michigan State Highway Department for furnishing the data for this article.

Summary of Findings

At intersections where stop-and-go signals were installed, the number of accidents increased nearly one-fourth. It was found that the simpler the intersection, the greater the increase in accidents. For example, simple three-leg intersections showed a gain in accidents of 78 percent, whereas five- and six-leg and other complex types showed a decline of 47 percent. In the case of flashing beacon installations, the number of accidents decreased regardless of the type of intersection, the range being 21 to 37 percent and averaging 26 percent.

The number of persons injured decreased by one-fifth at intersections where stop-and-go signals were installed and by one-half in the case of flashing beacon installations. The number of fatalities also decreased after either of the two types of signals were installed.

After installation of stop-and-go signals, the number of rear-end, head-on, and side-swipe collisions increased 200, 157, and 74 percent, respectively. Only angle and miscellaneous types of collisions decreased. The installation of flashing beacons resulted in a nearly uniform reduction of about one-fourth for each type of collision.

In the investigation of the effect of light and weather conditions on the accident rate following stop-and-go signal installation, it was found that the number of accidents during inclement weather increased fourfold as compared with accidents during all-weather conditions. This relation applied to both daytime and nighttime driving. After installation of flashing beacons, daytime accidents during inclement weather as well as during all-weather conditions decreased by one-fifth. On the other hand, nighttime accidents during inclement weather decreased by one-sixth as compared with a decrease of one-third for all-weather conditions.

At 38 intersections where traffic volume data were available, it was found that the greatest reduction in accident rates occurred at the higher volume intersections for stop-and-go signals and at the lower volume intersections for flashing beacons.

Accident Experience

At 39 intersections where stop-and-go signals were installed, the number of accidents increased 23 percent (table 1 and fig. 1). On the other hand, at 50 intersections where flashing beacons were installed, a 26-percent reduction in accidents was recorded.

The greatest increase in the number of accidents after stop-and-go signals were installed was observed at three- and four-leg undivided intersections (table 1 and fig. 2). The four-leg divided intersections showed practically no change, and at the more complex five- and six-leg intersections the number of accidents declined by 47 percent.

Flashing beacon installations resulted in a reduction in the number of accidents ranging from 21 percent at four-leg undivided intersections to 37 percent at five- and six-leg intersections. Compared with stop-and-go signals, this is a relatively narrow range and no general pattern was evident among the four types of intersections.

Both types of signal installations brought about a reduction in the number of persons injured: 20 percent for stop-and-go signals and 50 percent for flashing beacons. Fatal

Table 1.—Number of accidents, injuries, and fatalities before and after traffic signal installation at 89 intersections

Number of intersections	Type of intersection	Number of accidents				Number of injuries				Number of fatalities			
		Before signal installation	After installation of—			Before signal installation	After installation of—			Before signal installation	After installation of—		
			Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change [1]
3	3-leg	37	66	------	78	27	26	------	[2]—4	3	0	------	------
13	3-leg	114	------	78	—32	78	------	34	—56	3	------	4	------
23	4-leg, undivided	182	293	------	61	145	174	------	20	7	4	------	[2]—43
29	4-leg, undivided	219	------	174	—21	176	------	94	—47	21	------	6	—71
9	4-leg, divided	158	165	------	[2]4	136	91	------	—33	2	1	------	------
5	4-leg, divided	50	------	35	—30	42	------	16	—62	2	------	2	------
4	5- and 6-leg and other types	85	45	------	—47	70	13	------	—81	2	2	------	------
3	5- and 6-leg and other types	19	------	12	[2]—37	15	------	11	[2]—27	2	------	1	------
39	All types	462	569	------	23	378	304	------	—20	14	7	------	—50
50	All types	402	------	299	—26	311	------	155	—50	28	------	13	—54

[1] Too few data to compute percentage change, except for 4-leg undivided and all types of intersections.
[2] Percentage change is not statistically significant.

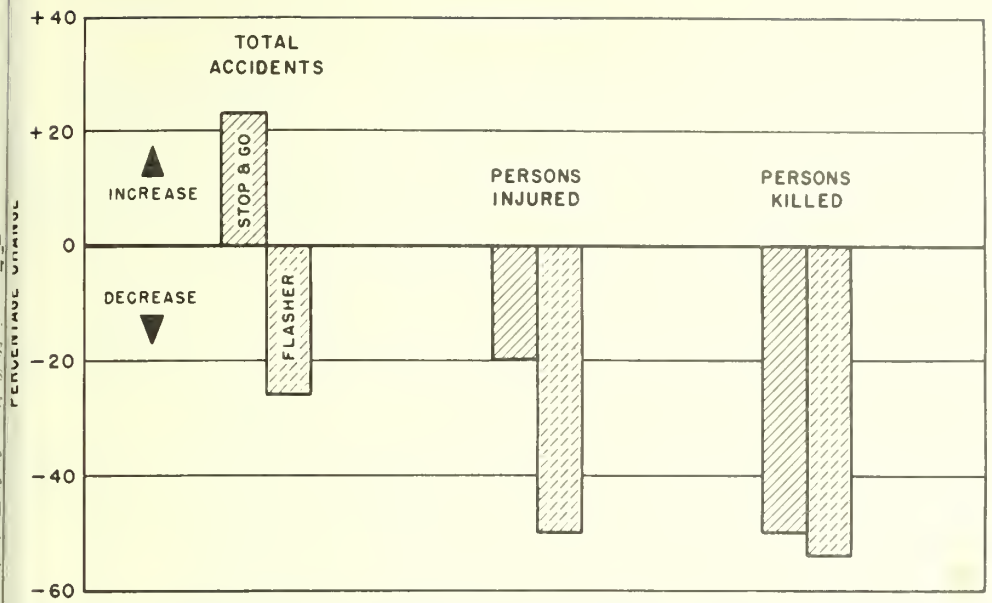


Figure 1.—Percentage change in the number of accidents, injuries, and fatalities after installation of traffic signals.

ties also decreased after installing either ype of signal, but no definite conclusions hould be drawn from this experience be- ause of the limited sample.

After stop-and-go signals were installed, he number of persons injured remained airly constant at three-leg intersections; in- reased slightly at four-leg undivided inter- ections; decreased moderately at four-leg ivided intersections; and decreased substan- ially at five- and six-leg and other complex ypes of intersections. This trend was quite imilar to that found for the number of ac- idents. At flashing beacon installations here was a general decrease in the number f persons injured at all types of intersec- ions, but no definite pattern was evident.

The data developed in this study indicate hat stop-and-go signals are less effective rom the safety standpoint at simple types f intersections, but for the more complex ypes they may be desirable. It appears that reater consideration should be given to the se of flashing beacons at three- and four-leg ntersections where some type of signal is equired.

Accident Exposure Considered

Up to this point exposure to accidents at he various intersections has not been con- idered. At 38 intersections where traffic olume data were collected, it was found that olumes increased 11.1 percent after install- ng stop-and-go controls and 11.5 percent fter installing flashing beacons. These per- entage increases applied generally to the ifferent types of intersections studied.

Accident, injury, and fatality rates based n 100 million vehicles entering the intersec- ions are shown in table 2. In general, the rends observed in table 2 for the 38 intersec- ions, after considering the rate of exposure o accidents, were similar to those shown in able 1 for all 89 intersections. There is one mportant exception, however. In the case of lashing beacon installations, the percentage decrease in the number of accidents remained fairly constant for the 50 intersections re- ported in table 1, regardless of the type of intersection studied, but for the 25 intersec- tions reported in table 2 the percentage de- crease became progressively greater with intersection complexity. For stop-and-go sig- nal installations, the relative changes in the number of accidents followed a similar trend in both tables 1 and 2. Whether the traffic volume factor is considered or not, it is im- mediately evident that stop-and-go signals are very effective in reducing accidents at the most complex types of intersections.

Traffic volumes at stop-and-go signal install- lations were on the average two or three times greater than the volumes at intersec- tions having flashing beacons. In order to provide a meaningful comparison of the effec- tiveness of stop-and-go signals and flashing beacons, intersections with like traffic volumes were selected. Since four-leg undivided intersections predominated in the study, the comparison has been narrowed to this type in table 3. By grouping the 4 lowest traffic volume intersections having stop-and-go sig- nals and the 4 highest volume intersections having flashing beacons, it was possible to select 2 groups of 4 intersections for which the average daily volumes approximated 16,000 vehicles.

The percentage change in accident and in- jury rates was computed for these eight intersections and for all four-leg undivided intersections (table 3). It is seen that install- lation of stop-and-go signals resulted in a substantial increase in the accident rate and a somewhat smaller increase in the injury rate. Flashing beacons, however, brought about a slight decline in the accident rate and a substantial decline in the injury rate. The pattern is the same whether the 8 intersec- tions or all 22 intersections are compared.

In a further attempt to determine the effect of traffic volumes on accident rates, the stop- and-go controlled intersections and those with flashing beacons were each divided into two groups: intersections with traffic volumes be- low the average for the group and those above the average (table 4). Averages for the two groups were 20,200 ADT for stop-and-go signals, and 8,000 ADT for flashing beacons.

At stop-and-go controlled intersections, the

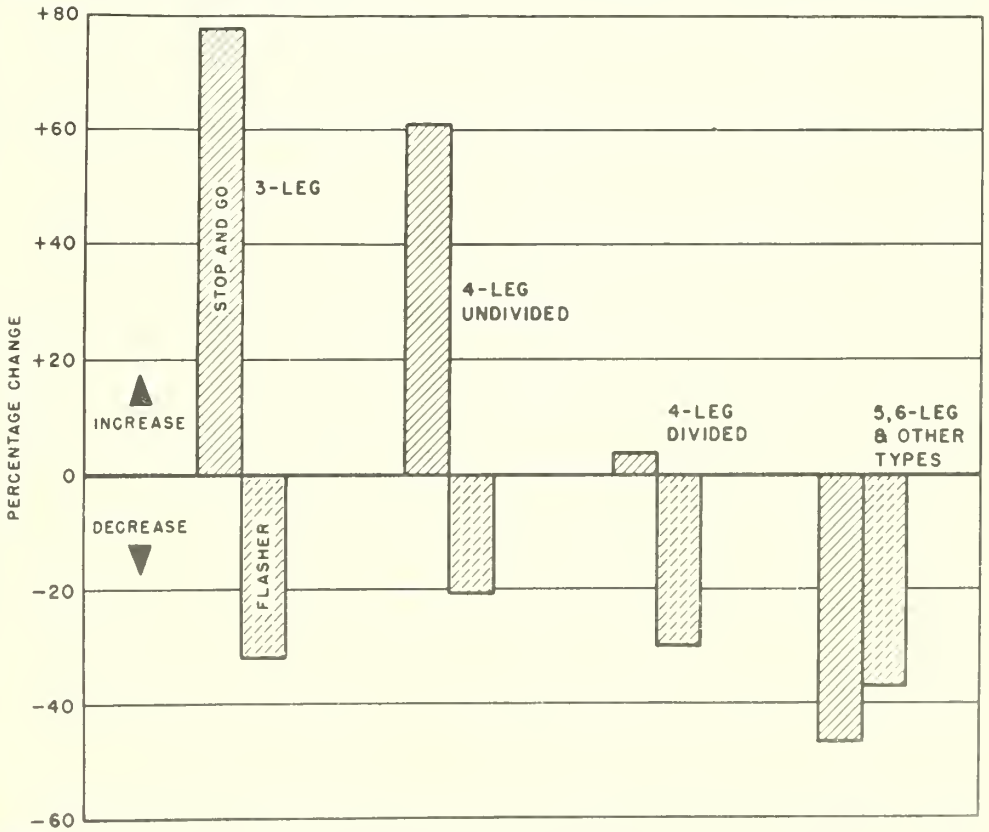


Figure 2.—Percentage change in the number of accidents, by type of intersection, after installation of traffic signals.

Table 2.—Number of accidents, injuries, and fatalities per 100 million vehicles entering before and after traffic signal installations at 38 intersections

Number of intersections	Average daily traffic	Type of intersection	Number of accidents per 100 million vehicles entering intersections				Number of injuries per 100 million vehicles entering intersections				Number of fatalities per 100 million vehicles entering intersections			
			Before signal installation	After installation of—			Before signal installation	After installation of—			Before signal installation	After installation of—		
				Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change [1]
2	11,800	3-leg	172	297	------	73	179	116	------	−35	20.7	0	------	------
5	6,700	3-leg	190	------	178	−6	108	------	94	−13	6.3	------	5.6	------
7	20,000	4-leg, undivided	130	199	------	53	62	76	------	23	1.6	1.5	------	------
15	8,200	4-leg, undivided	172	------	150	−13	134	------	74	−45	22.6	------	6.1	------
3	27,200	4-leg, divided	135	114	------	−16	118	29	------	−75	2.8	0	------	------
2	7,800	4-leg, divided	373	------	290	−22	280	------	157	−44	0	------	24.1	------
1	16,900	5- and 6-leg and other types	412	129	------	−69	471	81	------	−83	29.4	16.1	------	------
3	9,000	5- and 6-leg and other types	188	------	101	−46	150	------	92	−39	19.8	------	8.4	------
13	20,200	All types	153	182	------	19	117	67	------	−43	5.9	1.5	------	−75
25	8,000	All types	193	------	160	−17	143	------	87	−39	17.6	------	7.7	−56

[1] Too few data to compute percentage change for the different types of intersections.

accident rate increased 32 percent at the 8 intersections with below-average traffic volumes, and decreased 2 percent at the 5 intersections with above-average volumes. A similar comparison for flashing beacon installations showed a 22-percent decrease in the accident rate for 17 intersections having below-average traffic volumes and a 10-percent decrease for the 8 intersections with above-average volumes. Injury rates followed a declining trend, irrespective of traffic volumes or type of signal. Accident data reported in table 4 indicate that flashing beacons should be employed at low traffic volume intersections and stop-and-go signals at high volume intersections.

Traffic Signals Affect Types of Collisions

Types of collisions and the percentage change before and after installing stop-and-go signals and flashing beacons are reported in table 5 and illustrated in figure 3. Following installation of stop-and-go signals at 39 intersections, 3 types of collisions increased substantially: Rear-end collisions increased 200 percent; head-on collisions, 157 percent; and side-swipe collisions, 74 percent. Only angle collisions and other miscellaneous accidents showed a decrease. The 50 intersections with flashing beacons presented an entirely different picture. All types of collisions declined, ranging from 18 percent for rear-end collisions to 32 percent for the head-on type.

In the before period, angle collisions were dominant, but after installing stop-and-go signals, rear-end collisions ranked highest. However, after flashing beacons were installed, angle collisions still predominated, but there was a 29-percent decline in the number.

Since stop-and-go signals tend to reduce angle collisions, it follows that where this type of accident is a substantial percentage of the total, stop-and-go signals are more likely to be effective. Traffic engineers in the past have used this criteria in deciding whether a traffic control signal should be installed at a particular intersection.

Light and Weather Conditions

Table 6 shows that after stop-and-go signals were installed, the number of accidents increased regardless of light or weather conditions, with the greatest increase occurring during periods of inclement weather. During daylight hours, such accidents increased 126 percent as compared with an increase of only 32 percent for all-weather conditions. Similarly, accidents occurring at night during inclement weather increased 41 percent, as compared with an increase of only 11 percent for all-weather conditions.

With flashing beacons, the number of accidents decreased under all-weather conditions, both day and night. During daylight hours and inclement weather, the number decreased 20 percent as compared with an identical decrease for all-weather conditions. At night and during inclement weather, the number of accidents decreased 16 percent or about half as much as during all-weather conditions.

These comparisons point to the accident reduction benefit that can result from converting stop-and-go signals to flashing signals during inclement weather. More consideration should be given to this type of operation where signal controls are flexible enough to permit the conversion.

Established Policy on Traffic Control Devices

The Manual on Uniform Traffic Control Devices for Streets and Highways [2] provides that the total vehicular volume entering an

[2] Published by the Public Roads Administration, Washington, 1948 (also 1954 revisions supplement).

Table 3.—Percentage change in the number of accidents and injuries per 100 million vehicles entering after installing traffic signals at 4-leg undivided intersections

Comparison	Average daily traffic	Type of signal installed	Percentage change after installing signal	
			Accident rate	Injury rate
4 intersections with lowest ADT	15,800	Stop-and-go	81	52
4 intersections with highest ADT	16,600	Flashing beacon	−7	−66
All 7 intersections	20,000	Stop-and-go	53	23
All 15 intersections	8,200	Flashing beacon	−13	−45

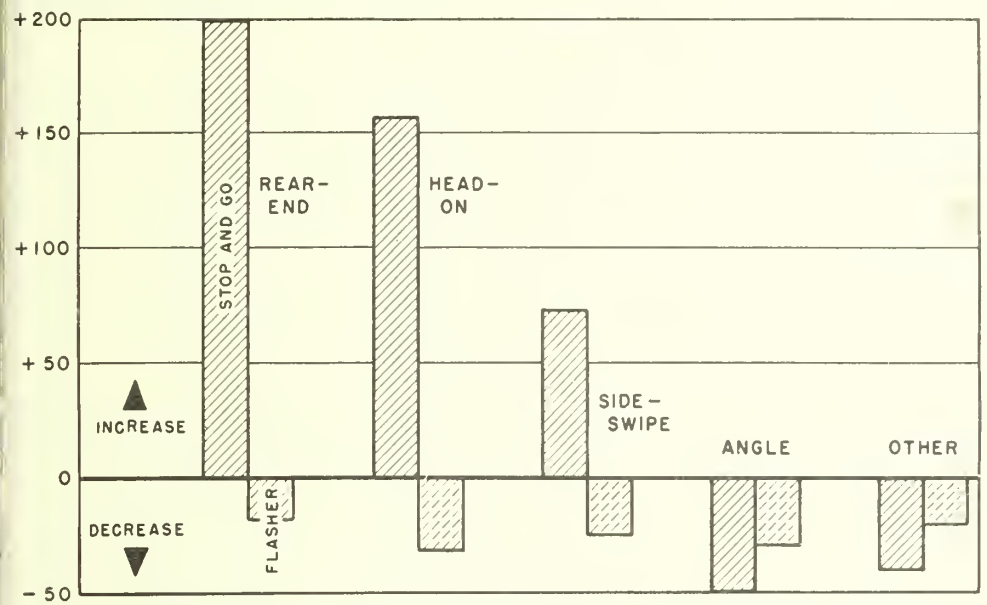
Table 4.—Number of accidents and injuries per 100 million vehicles entering before and after traffic signal installations at 38 intersections, grouped according to below average and above average daily traffic volumes

Number of intersections	Average daily traffic	Number of accidents per 100 million vehicles entering intersections				Number of injuries per 100 million vehicles entering intersections			
		Before signal installation	After installation of—			Before signal installation	After installation of—		
			Stop-and-go signal	Flashing beacon	Percentage change		Stop-and-go signal	Flashing beacon	Percentage change
8	Below 20,200	172	227	------	32	151	95	------	−37
5	Above 20,200	131	129	------	−2	77	34	------	−56
17	Below 8,000	285	------	223	−22	203	------	126	−38
8	Above 8,000	125	------	112	−10	98	------	56	−43
13	20,200	153	182	------	19	117	67	------	−43
25	8,000	193	------	160	−17	143	------	87	−39

able 5.—Number of accidents, by type of collision, before and after traffic signal installations at 89 intersections

Manner of collision	Accidents before signal installation	Accidents after installing **stop-and-go signal**	Percentage change	Accidents before signal installation	Accidents after installing **flashing beacon**	Percentage change
Rear-end	96	288	200	83	68	−18
Head-on	23	59	157	37	25	−32
Side-swipe	38	66	74	65	49	−25
Angle	259	128	−51	175	124	−29
Other	46	28	−39	42	33	[1] −21
All collisions	462	569	23	402	299	−26

[1] Percentage change is not statistically significant.



'igure 3.—Percentage change in the number of accidents, by type of collision, after installation of traffic signals.

ntersection from all approaches must exceed 50 vehicles per hour for each of 8 hours of n average day in urban areas and 500 vehicles per hour for each of 8 hours of an verage day in rural areas in order to justify top-and-go signals. An equivalent 24-hour raffic volume to satisfy these conditions would probably be about 15,000 vehicles per ay in urban areas and 10,000 vehicles per ay in rural areas.

Table 4 shows that stop-and-go signals installed at these volumes have resulted in an ncrease in accident rates. Table 3 shows this ven more strikingly for four-leg undivided ighways. The 4 lowest volume stop-and-o intersections with an average daily traffic olume of 15,800 vehicles showed an increase f 81 percent in the accident rate after the ignals were installed. The injury rate also ncreased. By way of contrast, both tables how that installation of flashing devices at hese low volumes resulted in decreases in oth accident and injury rates. It seems pparent, therefore, that consideration should e given to greater use of flashing beacons t intersections that barely qualify for stop-nd-go signals according to provisions of the nanual. The manual does state that stop-nd-go signals should be operated as flashing ed and flasbing yellow signals for the side treet and main street, respectively, if the raffic volume falls below 50 percent of the required minimums during two or more consecutive hours of the day. The present study provides additional support for this policy.

Throughout the article, the number of accidents, injuries, and fatalities have been expressed as a percentage relation of the experience before and after installing stop-and-go signals or flashing beacons. In order to test statistically the significance of the percentage changes that have occurred through modification of a highway intersection, two techniques have been developed by R. M. Michaels. His article titled *Two Simple Techniques for Determining the Significance of Accident-Reducing Measures* appears on pages 238–239 of this issue.

In testing for significance, the investigator seeks to determine whether a percentage change of a given magnitude is great enough to be attributed to the change introduced (e.g., installation of a traffic signal) or whether it is due to chance.

The two methods developed by Michaels, one labeled as a liberal test and the other as a conservative test, are based on the chi-square and the Poisson distributions. To make use of Michaels' methods of evaluation, the reader should refer to figure 1 on page 238. The percentage changes in the number of accidents, injuries, and fatalities reported in the tables of this article may be compared with the two curves shown in figure 1 of Michaels' article. In doing so, it will be observed that the relative changes reported in the tables for frequencies of accidents and injuries are, for the most part, statistically significant, based on a 5-percent probability level. In effect, this means that a percentage reduction in accidents of a given magnitude could not have occurred by chance more than once in 20 times. Percentage changes that are not considered statistically significant by either test are footnoted in the tables.

Recommendations

While more precise and complete information is needed to determine the conditions under which stop-and-go signals or flashing beacons may be most effective, the data presented here indicate that more widespread use of flashing beacons would be desirable at the simpler types of intersections.

A general and large-scale study of the relation between various design features of intersections and accidents should be undertaken to determine what type of traffic control device should be employed at a given intersection that would not only aid the movement of traffic but also provide maximum safety.

Flashing signal operation of certain stop-and-go signals during inclement weather seems to offer a possible method for reducing accidents. In determining when such signal operation should be used, the intersection accident and capacity factors should be fully considered.

Table 6.—Number of accidents, according to light and weather conditions, before and after traffic signal installations at 89 intersections

Light and weather conditions	Accidents before signal installation	Accidents after installing **stop-and-go signal**	Percentage change	Accidents before signal installation	Accidents after installing **flashing beacon**	Percentage change
Daytime:						
Clear	171	186	[1] 9	132	110	−17
Cloudy	59	81	37	58	43	−26
Rain, snow, or fog	39	88	126	45	36	[1] −20
All conditions [2]	269	355	32	235	189	−20
Nighttime:						
Clear	104	106	[1] 2	91	63	−31
Cloudy	51	55	[1] 8	34	15	−56
Rain, snow, or fog	34	48	41	32	27	[1] −16
All conditions [2]	189	209	11	157	105	−33

[1] Percentage change is not statistically significant.
[2] The number of accidents in this table are slightly less than those given in other tables because light and weather conditions were not known in every case.

Two Simple Techniques for Determining the Significance of Accident-Reducing Measures

BY THE DIVISION OF TRAFFIC OPERATIONS
BUREAU OF PUBLIC ROADS

Reported by RICHARD M. MICHAELS
Research Psychologis

WHENEVER an attempt is made to reduce accidents on a section of highway through modification of some of the accident-producing features, the question arises as to whether an observed reduction is due to anything more than chance alone. The most common approach to evaluation has been to institute improvements on sections of highways where a large number of accidents have occurred. Then after modification, reductions in accidents are reported as a percentage relationship of the before and after experience. At this point it must be determined whether the observed percentage reduction is great enough to be ascribed to the change introduced or whether it is due to chance factors. An attempt is made in this article to describe two simple relations that are available for determining the significance of a percentage reduction in accidents.

Accident Study Requirements

In order to make a comparison on a before-and-after basis, one must be certain that the two study periods are reasonably comparable. The following factors must be considered in the evaluation: (1) Some measure of vehicle-miles is needed for both the before and after periods in order to equate accident exposure; (2) the traffic volumes for each of the two periods should be approximately the same, for marked differences in volumes will affect the accident experience;[1] (3) the composition of the traffic on the study section should be unchanged during each of the two periods as this, too, may influence accident experience; and (4) since the fatal accident rate has consistently tended to decrease over the years, the accident totals in the after period should be corrected for any existing trends.

The accident figure used for analysis should be the total number of accidents by type, either fatality, injury, or property damage. The total number of fatalities or the total number of injuries should not be used whenever the number of accidents is less than 50.[2] The restriction in considering accidents by type reflects the fact that the number o deaths or injuries will usually be determine by factors peculiar to the individual accident Thus, the number of occupants of the vehicle its speed, or the geometry of collision may b the variables that determine the number o casualties. These are usually independent o the highway and any modification mad thereon. Therefore, analyzing the frequency of accidents by type should yield a more pre cise measure of the effects of highway modifications.

[1] *The Interstate Highway Accident Study,* by Morton S. Raff. PUBLIC ROADS, vol. 27, No. 8, June 1953; also Highway Research Board Bulletin 74, 1955, pp. 18–45.

[2] Although the selection of a sample size of 50 is arbitrary, it should generally minimize any bias that might be imposed by the rare accident in which a large number of fatalities or injuries occurred. The probability of more than five deaths or injuries occurring in a single accident is very low. Consequently, with a sample size of 50 or more, these single cases will inflate the accident frequency rarely more than 10 percent.

Application of Poisson Distribution

Accidents are rare events in time, distance or among individuals. For example, th probability of a fatal accident is only abou 5 chances in 100 million miles of driving The same low probability exists whether th reference is thousands of vehicles or fre quencies of accidents among a group o people. An applicable statistical model is th Poisson distribution which is an approxima tion to the binomial density function whe the probability of the event is low and th population in which it occurs is large.

In general, the assumptions of the Poisso distribution are met in most accident situa tions. Several studies have shown that thi

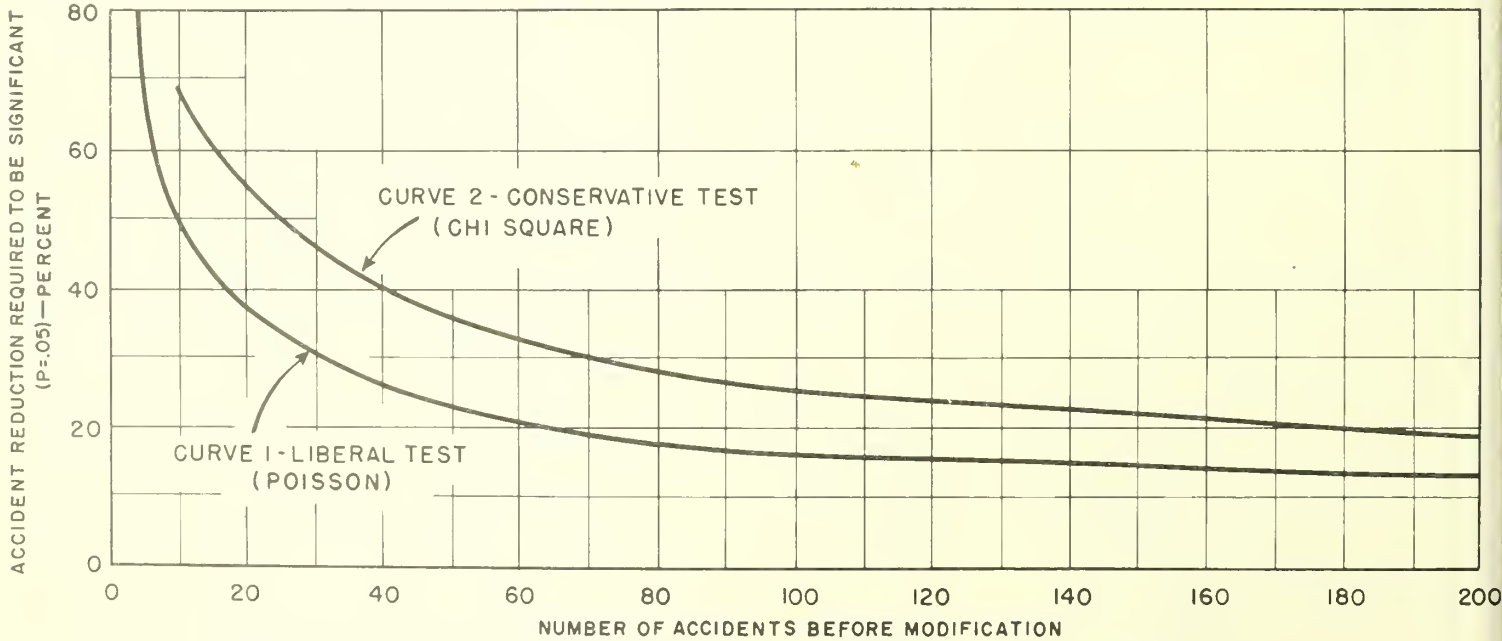


Figure 1.—Curves for determining the statistical significance of accident-reducing techniques.

model fits traffic accident data very well.[3] Therefore, one way to approach the problem of determining the significance of a percentage reduction in accidents is to assume that the observed data are a sample from a Poisson distribution.

If the before time period accident frequency is used as the expected value of the Poisson distribution, then it is possible to determine the percentage decrease in accidents that is statistically significant. Given the number of accidents before modification, the solution of the Poisson distribution leads to curve 1 which is illustrated in figure 1. The curve shows the percentage reduction in accidents that can be attributed to something more than chance. To illustrate the application of figure 1, assume that 40 accidents occurred on a given section of highway before modification. Reading from the graph (curve 1), it is observed that a 26-percent reduction in the number of accidents would indicate a significant change at a 5-percent probability level. Thus, a reduction in accidents due to chance would occur only once in 20.

Curve 1 is a liberal estimate of significance, and it must be interpreted carefully. The number of accidents in the before time period is assumed to be the true mean of the Poisson distribution. Since the number of accidents varies from year to year, taking any one year as the expected value of the population is arbitrary. Consequently this estimate of the mean may be in considerable error. It is especially suspect when the number of accidents in the before time period is abnormally high or low. Averaging over several comparable years is probably the best compromise. Within this limitation the test requires the smallest percentage reduction for obtaining statistical significance.

Application of Chi-Square Test

In order to make a more conservative estimate, an alternative is the chi-square test. By using this test it is possible to determine whether the two samples differ significantly; that is, whether the number of accidents occurring after corrective measures are instituted is reliably less than the number before. Curve 2 in figure 1 shows the percentage reduction in accidents that is significant as a function of the number of accidents occurring before improvements are made. The test assumes that the highway sections remain the same or can be adequately corrected from year to year.

[3] *Poisson and Traffic,* by Daniel L. Gerlough and André Schuhl. Eno Foundation for Highway Traffic Control, Saugatuck, Conn., 1955, 75 pp.

The two curves represent two limits which can be applied to determine whether there is a reliable reduction in accidents. Curve 1 may be used to minimize the chance of calling a reduction not significant when in fact it is, and curve 2 may be used to minimize the chance of calling a reduction significant when it is not. In general, curve 2 should be used where there is limited before data. If accident data for several years before modification are available, and show a variation of no more than 20 percent from year to year, they may be averaged. Then the after time period may be compared with this average and curve 1 used to determine whether the reduction is statistically significant.

Testing for Significance

In order to demonstrate the use of the curves, four examples from two studies have been selected. In one study, the effect of traffic control devices on accidents at different types of intersections was investigated.[4] It was found that prior to installation of flashing beacons at five- and six-leg intersections, 19 accidents were recorded. After installation of flashing beacons, the number of accidents decreased 37 percent. Figure 1, curve 1, shows that for this size sample, it is reasonable to conclude that the flashing device did not significantly reduce accidents and the reduction could have been due to chance alone.

At four-leg intersections on divided highways, it was found that after installation of flashing beacons the number of accidents were reduced from 50 to 35, or 30 percent. Reference to figure 1 shows that for a before value of 50 accidents, a 30-percent reduction is significant according to the liberal test, but it is not according to the conservative test. Since only 1 year of before data is available, the conservative test is preferred. Therefore, it is reasonable to conclude that the flashing device did not significantly reduce accidents at these intersections.

One of the remaining two examples of accident experience, selected from a second study,[5] indicated a significant reduction in accidents according to the liberal test, whereas the other example met the requirements of both tests.

At an intersection of a four-lane divided highway and a two-lane highway, there was an average of 27 accidents over a 2-year period. Most of these accidents were due to left-turn movements from the major to the minor highway. In an attempt to reduce accidents, a left-turn lane was added to the major highway and a left-turn arrow was added to the traffic signal. After these modifications there were 16 accidents per year, or a 41-percent reduction. Figure 1 shows that this is a significant reduction using the liberal test but not using the conservative test. Since the data are averages for 2 comparable years, the liberal test is the logical one to use. On this basis it is reasonable to conclude that the traffic engineering measures significantly reduced accidents at this intersection.

At one point on a four-lane divided highway, drivers had to negotiate a sharp curve. Over a 3-year period there was an average of 17 accidents per year. In an attempt to reduce accidents, highway lighting was added and also road-edge delineators. After the changes, there were 5 accidents per year, or a 70-percent reduction. With reference to figure 1, such a reduction is statistically significant by both tests. Therefore, it is reasonable to conclude that the engineering changes did cause a statistically reliable reduction in accidents.

Statistical Limitations

A word of caution is in order concerning the use of the curves. Both tests require fulfillment of certain statistical assumptions to be wholly valid. There is no way to prove that these assumptions are met in any given field situation. Accident processes are not constant in time, and the factors influencing their change are poorly understood at best. Furthermore, there is never good control over all the variables influencing changes in accident incidence. Populations of drivers change, as do populations of vehicles. In addition, several accident reduction measures are always operating, e.g., enforcement and education. It is therefore rarely possible to eliminate the influence of all such forces and ascribe an observed reduction unequivocally to some single specific measure.

The curves shown may be used as an estimate of the percentage reduction in accidents that is required to achieve statistical significance. In using these curves the limitations inherent in any field study should be recognized. Where techniques or measures do appear to show a significant influence on accident incidence, a more intensive evaluation of them may be required. This can and should be done with emphasis given to careful control and more precise measurements.

[4] *Traffic Signals and Accidents in Michigan,* by David Solomon. See pp. 234–237 of this issue.

[5] *Reducing Accidents by Traffic Engineering.* Virginia Department of Highways, Richmond, 1957, 31 pp.

Estimated Travel by Motor Vehicles in the United States, 1957

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH,
Highway Research Engineer

THE average motor vehicle traveled 9,571 miles in 1957, almost one-half of it in cities, and averaged 12.47 miles per gallon of fuel. Total motor-vehicle travel in 1957 amounted to 647 billion vehicle-miles. For 1958 the total is estimated at 665 billion, and nearly 700 billion is forecast for 1959.

Of the 1957 travel, 40 percent was on main rural roads, which constitute 16 percent of the Nation's 3.4 million miles of roads and streets. Another 46 percent of the travel was on urban streets, which include only 11 percent of the total mileage. Local rural roads, comprising 73 percent of all mileage, carried 14 percent of the travel.

Passenger cars represented 83 percent of the vehicles and performed 82 percent of the travel in 1957. The average passenger car traveled 9,391 miles and consumed 652 gallons of fuel at a rate of 14.40 miles per gallon. Trucks and combinations accounted for 16 percent of the vehicles and 17 percent of the travel. The average truck or combination traveled 10,328 miles, but consumed twice as much fuel as the average passenger car, 1,302 gallons, at a rate of 7.93 miles per gallon. Buses accounted for the remaining 1 percent of the vehicles and 1 percent of the travel.

Table 1.—Estimate of motor-vehicle travel in the United States, by vehicle types, in the calendar year 1957

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [1]	203,542	71,492	275,034	254,371	529,405	56,375	9,391	36,769	652	14.40
Buses:										
Commercial	942	156	1,098	1,943	3,041	85	35,776	641	7,541	4.74
School and nonrevenue	552	551	1,103	249	1,352	179	7,553	184	1,028	7.35
All buses	1,494	707	2,201	2,192	4,393	264	16,640	825	3,125	5.32
All passenger vehicles	205,036	72,199	277,235	256,563	533,798	56,639	9,425	37,594	664	14.20
Trucks and combinations	54,960	18,110	73,070	40,136	113,206	10,961	10,328	14,271	1,302	7.93
All motor vehicles	259,996	90,309	350,305	296,699	647,004	67,600	9,571	51,865	767	12.47

[1] Includes taxicabs, motorcycles, and light trailer combinations pulled by passenger cars.

Travel Estimate Procedures

The travel data in table 1 are based on comprehensive studies made by the States in connection with the highway cost allocation study undertaken by the Bureau of Public Roads.[1] In that study, travel in municipalities of less than 5,000 population is included with rural travel, in accordance with the rural-urban classification of the Federal-aid systems. In table 1, however, all municipal travel is shown as urban. This is in accordance with the motor-vehicle travel data that have been published annually by the Bureau of Public Roads for a number of years.[2]

During 1957, more than 365,000 traffic volume counts of 16 hours' duration or longer were obtained either manually or with machines of various types. Permanently located counters are operated continuously at more than 1,500 locations throughout the United States. The continuous counts, when properly grouped and related to road sections having similar traffic characteristics, provided factors by which 24- or 48-hour counts were used to calculate the average traffic for the year.

Traffic counts in which the numbers of vehicles of each type are determined or classifications counts, as they are called, were obtained manually since no machine has yet been developed to obtain such information economically. To calculate reliable values for annual travel, observations were made during all seasons of the year, at all hours of the day, and on all days of the week. Approximately 28,700 classification counts were obtained during 1957, making it possible to compute the annual vehicle-miles of travel for each visual class of vehicle. These estimates were prepared by the States and supplied to the Bureau of Public Roads.

The new studies have made possible a broader, more reliable base for travel estimates. However, the total travel of 647,004 million vehicle-miles, calculated from the new estimate base, was very close to the total travel determined from the former trend procedures, being only 0.7 percent higher.

Significance of New Estimate Base

The new estimates indicate that the proportion of truck travel as well as the proportion of travel on local rural roads was previously overestimated. While the 1957 figure of 113,206 million vehicle-miles for all trucks and combinations is 2.5 percent below the 116,100 million vehicle-miles reported for 1956, the new trend data indicate that there was actually an increase of 1 to 2 percent. Similarly, 1957 estimates show that local rural roads carried 5 percent less traffic than was reported in previous years, and main rural roads and urban streets carried larger percentages; but from the new trend data it appears that increases in travel of 2 percent or more occurred on each of the three classes of roads and streets.

Values for fuel consumption by vehicle type have been recalculated on the basis of information developed for the *Highway Cost Allocation Study*, adjusted to 1957, and published registration data.[3] As a result, the average travel per gallon of fuel consumed for some vehicle types is slightly different from earlier 1957 published estimates. The new figures indicate a lower value of 7.35 miles per gallon for schoolbuses. The fact that this value is less than that for trucks and combinations is probably due to the stop-and-go driving characteristics of schoolbuses. The comparatively high mileage per gallon of fuel for trucks and combinations is due to the fact that panel and pickup trucks account for nearly half of all travel by trucks and combinations.

[1] *Third Progress Report of the Highway Cost Allocation Study*, H. Doc. 91, 86th Cong., 1st sess., 1959.

[2] See previous articles in PUBLIC ROADS magazine; the most recent article, for 1956, appears in vol. 29, No. 11, December 1957.

[3] *Highway Statistics 1957*, Bureau of Public Roads, table MV–1, p. 48.

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VOL. 30, NO. 11 DECEMBER 1959

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

The Pacific Highway (Interstate Route 5) south of Salem, Oregon.

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 11 *December 195*

C. M. Billingsley, *Editor*

IN THIS ISSUE

Circular Reinforced Concrete Columns Subjected to Direct Stress and Bending 211

Sampling Techniques Applicable to the Collection of Economic Data 216

Table: State Legal Maximum Limits of Motor-Vehicle Sizes and Weights Compared With AASHO Standards (see text on p. 255) . . 250

Factors Influencing Mass-Transit and Automobile Travel in Urban Areas 256

U.S. DEPARTMENT OF COMMERCE
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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

Circular Reinforced Concrete Columns Subjected to Direct Stress and Bending

Prepared for the Bureau of Public Roads
by A. ANTHONY TOPRAC [1]

Based on the American Association of State Highway Officials' bridge specifications for allowable loads, a procedure for designing eccentrically loaded round columns has been developed by the author. Under the enlarged highway construction program, bridge engineers will more frequently be confronted with the problem of designing round columns for bridge pier bents.

The principal benefit of this article will be derived from the method outlined in the text, which will assist bridge designers who may want to expand the graphs presented here or make similar graphs for different bridge column sections. Plotted curves are given for a range of 24- to 60-inch diameter columns in 6-inch intervals and for three quantities of reinforcing steel. In design problems for which the graphs are applicable, considerable tedious work will be avoided.

THE 1957 American Association of State Highway Officials' specifications (*1*) [2] on the design of eccentrically loaded columns state that for ratios of eccentricity to depth (diameter) not greater than 0.5, the allowable load could be calculated on the basis of uncracked section, and for ratios greater than 0.5 the design or the allowable load should be based on the theory of cracked sections.

Based on these specifications, a procedure for designing eccentrically loaded columns has been developed and is presented here. Curves have also been prepared for 24- to 60-inch diameter columns in 6-inch intervals for three percentages of steel in each case.

The problem of designing or investigating round columns subject to bending is more difficult than the same problem for rectangular columns. With the increased activity in highway construction, many engineers will be confronted with the design of round columns for bridge pier bents. To facilitate the design and investigation of such columns, a method is presented in the following discussion. The method hinges on the ability of the designer to guess at the position of the neutral axis; the designer then calculates the allowable stress and makes proper use of the expression $N/A_t \pm Mc/I_t$. Once the position of the neutral axis is checked, properties of the cross section such as r^2 and K are calculated and used to obtain allowable stresses.[3]

Approaches to Problem of Eccentrically Loaded Columns

The author found three methods which are used to solve the problem of eccentrically loaded columns. Although these methods are based on the same fundamental laws of equilibrium, on the surface they look different. Consequently, it is worthwhile to outline them in short form.

1. In the first of these methods, used by Boguslavsky (*2*), the effective portions of a cracked round section are a circular segment of concrete in compression and a circle of steel bars; some of them in compression, others in tension. The internal stresses acting on the transformed section vary elastically from f_s/n to f_s in straight-line proportions. The resultant C of the compressive stresses acting on the circular segment of concrete is represented by the volume of the ungula (wedge cut by a sloping plane from a circular prism), while the tensile resultant T of the tensile and compressive stresses acting on the concrete ring, which replaces the steel, is represented by the volume obtained by multiplying the area of the ring by the surface of the circular prism, intercepted between the horizontal and sloping planes. Formulas are used for C and T, for their moment arms from the axis of the column, and for the moments. The derivation of these formulas, which contain one-half the angle subtended at the center of the circle by the chord on the neutral axis, may be found in Peabody's *Design of Reinforced Concrete Structures* (*3*).

2. In the second method, presented by Dunham (*4*), an analysis of the stress diagram shows that the imaginary solid that represents the compression upon the concrete is an ungula of a circular cylinder (a wedge-shaped solid). In order to simplify the calculations, the longitudinal rods in the column are assumed to be replaced by a hollow steel shell or pipe which has the same cross-sectional area as that of all of the longitudinal reinforcement. This steel shell is subjected to compression upon part of its area and to tension upon the rest of its cross section. Therefore, the diagrams that represent the stresses upon these two parts of the ring will be like hollow ungulas or like wedges cut from a pipe. In order to have equilibrium, it is obvious that $\Sigma V=0$ and $\Sigma M=0$. Curves prepared by Dunham give the volume and the moment of ungula about the neutral axis for the concrete in compression. Similar curves give the volume and moment of wedge of hollow cylinder (for the steel) about the same neutral axis. Then, the desired allowable stresses are obtained by applying the two equations of equilibrium.

3. The third method, presented by Large (*5*), involves the following computations:

(1) Calculate trial stresses by $N/A_t \pm Mc/I_t$, assuming no area lost by cracking, and from the stress diagram determine the approximate depth of the crack.

(2) Compute corrected stresses by $N/A_t \pm Mc/I_t$, of the cracked section, using curves developed by Professor J. R. Shank for the properties of circular segments. The curves (*6*) showing these properties are presented in figure 1.

Approach Adopted

Of the three methods the last one was found to be relatively more practicable and was adopted in the preparation of curves plotting load versus eccentricity. To facilitate computations, figure 2 was prepared.

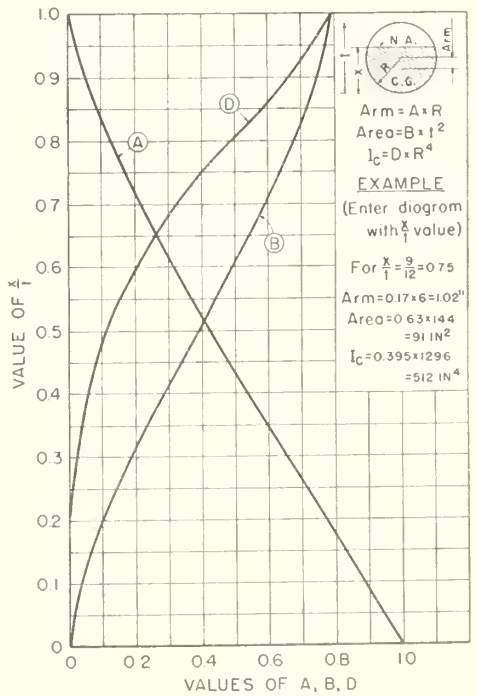
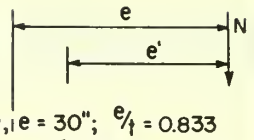


Figure 1.—Properties of segments of a circle. **(Courtesy of the American Concrete Institute.)**

[1] This article was prepared by Professor Toprac, Associate Professor of Civil Engineering, University of Texas, while employed during the summer months of 1958 by the Bureau of Public Roads, Texas Division office, Austin.

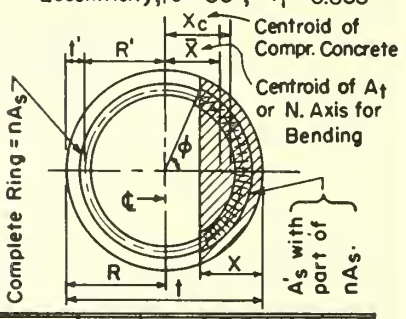
[2] Italic numbers in parentheses refer to the list of references on p. 245.

[3] Letter symbols used in this article are defined on pp. 244-245.



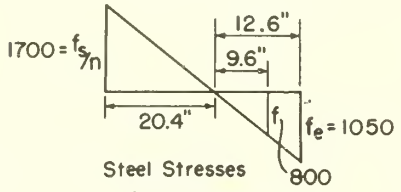
Given: $t = 36"$ $t' = 3"$ $d = 33"$ $A_s = 12.7 in.^2$ $A_g = 1018 in.^2$ $p = 0.0125$ $n = 10$

Eccentricity, $e = 30"$; $e/t = 0.833$



N.A. from Comp. Edge	$k = \frac{x}{t}$	Eff. Area of Concrete		Area of Steel			Transf. Area A_t	$\frac{N}{A_t}$	Coeff. A	X_c	Bending N.A. $\bar{X}$	$X_c - \bar{X}$
		Coeff. B	A_c	nA_s	$\lambda . A_s = A'_s$	A_{st}						
$X = 10.8$	0.30	0.20	259	127	$0.34 \times 127 = 43$	170	429	233	0.65	11.72	8.30	3.42
$= 11.5$	0.32	0.218	282	127	$0.354 \times 127 = 45$	172	454	220	0.624	11.25	8.20	3.05
$= 12.6$	0.35	0.25	324	127	$0.38 \times 127 = 48$	175	499	200	0.59	10.64	8.00	2.64

Moment of Inertias							Concr. C	$\frac{Ne'c}{I_t}$	$\frac{Ne'(R'+\bar{X})}{I_t}$	Combined Stresses		Calc. $X = d \times \frac{f_c}{f_c + f_s/n}$	$r^2 = \frac{I_t}{A_t}$	$K = \frac{t^2}{2r^2}$	f_o	f_e
Coeff. D	I_c	I_s	$A_c(X - \bar{X})^2$	$nA_s\bar{X}^2$	$A'_s(X' - \bar{X})^2$	$\sum = I_t$				f_c	f_s/n					
0.02	2,100	14,200	3,000	8,750	690	28,740	9.7	733	1760	966	1527	12.8				
0.025	2,530	14,200	2,600	8,550	720	28,600	9.8	746	1770	966	1550	12.7				
0.034	3,580	14,200	2,250	8,125	660	28,815	10.0	763	1755	963	1555	12.6	57.7	11.2	628	1105



$f_s = 17{,}000$

$f'_s = 2nf = 16{,}000$

$$N = 100{,}000 \times \frac{f_e}{f_c} = 100{,}000 \times \frac{1050}{963}$$

$$= 109{,}000 \text{ lbs.}$$

Formulas

$A_c = Bt^2$ $\qquad \bar{X} = \frac{\Sigma(\text{Areas}) \times (\text{dist.})}{A_t}$

$I_c = DR^4$ $\qquad X' = \text{c.g. dist. from } ℄ \text{ of } A' = 57.3\, R' \frac{\sin\phi}{\phi^\circ}$

$X_c = AR$

$\lambda = \phi^\circ/180$ $\qquad \cos.\phi = 1 - \frac{a}{R}$, where

$c = R - \bar{X}$ $\qquad a = X - t'$

For $X = 10.8"$

$\cos\phi = 1 - \frac{7.8}{15} = 0.48$

$\phi = 61.3^\circ$

$259 \times 11.72 = 3030$

$43 \times 12.3 = \frac{530}{3560}$

$\bar{X} = \frac{3560}{429} = 8.3"$

$X_c - \bar{X} = 11.72 - 8.30 = 3.42$

$X' - \bar{X} = 12.3 - 8.3 = 4.0$

$C = R - \bar{X} = 18 - 8.3 = 9.7"$

$e' = 30 - 8.3 = 21.7"$

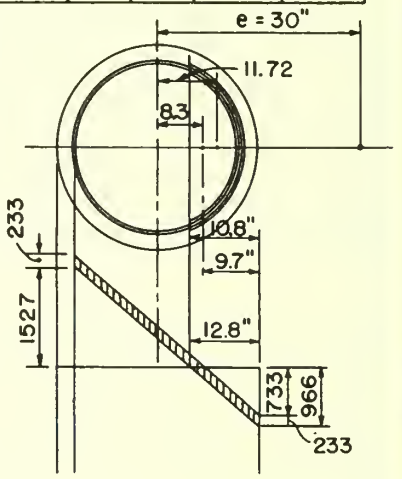


Figure 2.—Sample calculations for design graphs (figs. 3–6).

To solve for the allowable load a column could carry for a given e/t ratio, the steps are as follows:

1. Assume distance of neutral axis to extreme compression fiber in the concrete and calculate $k = x/t$.

2. Referring to figure 1, coefficients A, B, and D are obtained for the calculation of the distances x_c, A_c, and I_c.

3. The steel reinforcement is replaced by a ring of equivalent area nA_s and a circular segment in the compression side, equal to nA'_s in area. This last steel is considered because the AASHO specification 1.7.8(f) states that "In such cases the value of the compressive reinforcement may be taken as twice the value given by a straight-line relationship between stress and strain and the modular ratio, n . . ."

4. Find the direct stress N/A_t, assuming $N = 100{,}000$ pounds.

5. Find $\bar{x}$ for the neutral axis for bending only, by finding the center of gravity of the transformed area and calculating the moment of inertia of the transformed area to be used in the expression Mc/I.

6. Calculate the actual stresses for 100,000 pounds eccentric load,

$$f_c = \frac{Mc}{I_t} + \frac{N}{A_t} \text{ and } f_s/n = \frac{-M(R' + \bar{x})}{I_t} + \frac{N}{A_t}.$$

7. Calculate the allowable compressive stress f_c and check the stresses in the tension and compression steel that correspond to this calculated f_c.

8. The eccentric allowable load then is $N = 100{,}000 \times$ (allowable compressive stress/actual compressive stress).

The above procedure was used in the preparation of the curves given in figures 3–6.

Practical Illustrations

The following are illustrative examples which show the various steps just mentioned.

Example I.—Given a 36-inch outside diameter tied column with 10 No. 10 bars as longitudinal reinforcement, calculate the allowable compressive load applied 5 inches from the center of the column, if $f'_c = 3{,}000$ p.s.i. and $n = 10$.

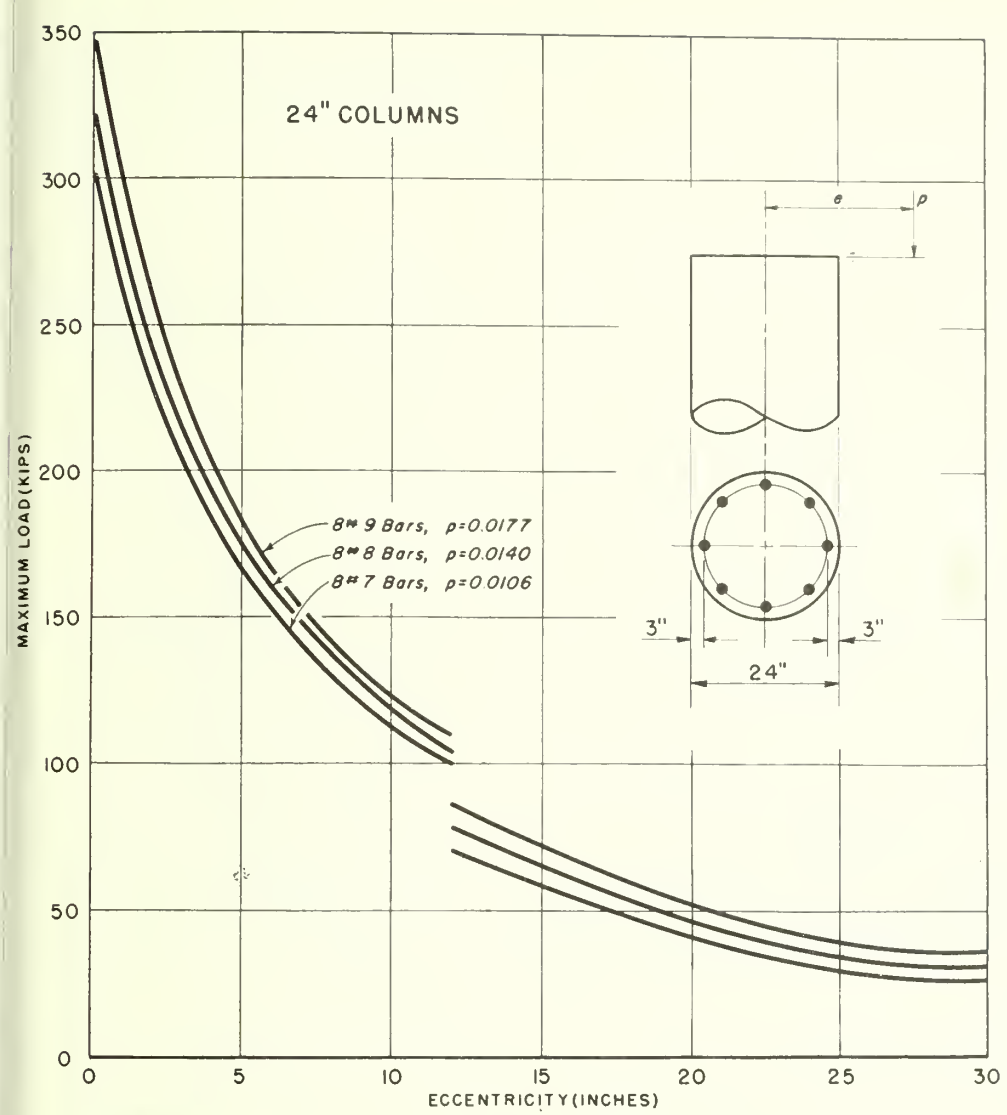


Figure 3.—Maximum load for 24-inch circular-tied columns in direct stress and bending.

Solution.—Since $e/t=5/36=0.139$ which is less than 0.5, use of the uncracked section is allowed by the specifications.

$P_t=0.8\ (0.225\ f'_cA_g+A_sf_s)=0.8\ (0.225\times 3{,}000\times 1{,}018+12.7\times 16{,}000)=0.8\ (686{,}000+203{,}000)=713{,}000$ pounds centrally applied load.

The transformed area, $A_t=1.018+9\times 12.7=1{,}133$ in.²

The concrete compressive stress, $f_a=713{,}000/1{,}133=628$ p.s.i.

Then $C_1=f_a/0.40f'_c=628/1{,}200=0.523$.

The moment of inertia of the transformed area, $I_t=I_c+I_s$. Where $I_c=\text{area}\times t^2/16=1{,}018\times 1{,}296/16=82{,}500$ in.⁴, and $I_s=\text{area}\times(t-2t')^2/8=115\times 30^2/8=13{,}000$ in.⁴ Thus, $I_t=82{,}500+13{,}000=95{,}500$ in.⁴

$r^2=I_t/A_t=95{,}500/1{,}133=84.0$, and $K=t^2/2r^2=1{,}296/2\times 84=7.70$.

$C_1K=0.523\times 7.70=4.03$, and $1+C_1K(e/t)=1+4.03\times 0.139=1.560$.

$N=P_t/1+C_1K(e/t)=713{,}000/1.560=457{,}000$ pounds.

Example II.—Same as example I, except $e=30$ inches.

Solution.—Since $e/t=30/36=0.833$ which is greater than 0.5, use method of cracked section. To facilitate calculations, figure 2 is used. Assume $kt=10.8$ inches and $k=0.30$. From figure 1 coefficients B, A, and D are found to be 0.20, 0.65, and 0.02, respectively. The compression concrete area then is $0.20\times 36^2=259$ in.² The equivalent concrete that replaces the reinforcing bars is a ring of $n\times A_s=10\times 12.7=127$ in.²

Since the AASHO specification 1.7.8(f) states that ". . . the value of the compressive reinforcement may be taken as twice the value given by a straight-line relationship between stress and strain and the modular ratio . . .", a circular segment of the equivalent concrete ring is added to the remaining area. A factor, λ, is calculated for this purpose based on the angle $\phi°$ (half of the angle subtended at the center of the column by chord which coincides with the assumed neutral axis). This angle is given by the expression $\cos\phi=1-a/R'=1-7.8/15=1-0.52=0.48$; then the angle $\phi=61.3°$. The amount of additional compressive steel is $\phi°/180\times nA_s=0.34\times 127=43$ in.², and all the transformed area of the cracked section becomes equal to 429 in.² Assuming the load on the column to be $N=100{,}000$ pounds, the direct compression stress is $N/A_t=233$ p.s.i.

The same transformed section is also in bending because of this load N, which is applied eccentrically. Bending takes place around a neutral axis (for bending only) which passes through the center of gravity of $A_t=429$ in.²

The center of gravity of the concrete area is 0.65×18 inches $=11.72$ inches from the center of the column. By taking the first moments of the areas, the bending neutral axis is found to be $\bar{x}=8.30$ inches away from the center of the column. The moment of the concrete is 3,030 in.³ and that of the additional compressive steel is 530 in.³ The moment arm of this steel is obtained by the expression, $x'=(57.3R')\sin\phi/\phi°=(57.3/61.3°)(15)(\sin 61.3°)=14\times 0.877=12.3$ inches.

To calculate bending stresses, the formula Mc/I is used where the moment of inertia $I=I_t$. The moment of inertia for the transformed area around the bending neutral axis, I_t, is calculated as follows:

The moment of inertia, I, of compressive concrete around its center of gravity is, $I_c=0.02\times 18^4=$	2, 100
Additional moment of inertia, I, to transfer above to neutral axis equals $259\times 3.42^2=$	3, 000
Moment of inertia, I, of the equivalent ring, $I_s=nA_s\times(2R')^2/8=127\times 30^2/8=$	14, 200
Additional moment of inertia, I, to transfer above to neutral axis equals $nA_s(\bar{x})^2=127\times 8.3^2=$	8, 750
Moment of inertia, I, of additional compressive steel equals $A'_s\,(x'-\bar{x})^2=43\times 4.0^2=$	690
Moment of inertia for transformed area around the bending neutral axis, $I_t=$	28, 740 in.⁴

The bending moment then is equal to $Ne'=100{,}000\times 21.7$ inches, and the extreme fiber concrete stress $Ne'c/I_t=733$ p.s.i.; the bending tensile stress equals 1,760 p.s.i.

The combined stresses are $733+233=966$ p.s.i. compression in the concrete and $-1{,}760+233=-1{,}527$ p.s.i. tension in the steel. However, these stresses require that $kt=12.8$ inches which is larger than the assumed 10.8 inches. Therefore a new value must be assumed and checked. This has been done twice and the values appear in figure 2. It will be noticed that the second guess of $x=kt=11.5$ inches resulted in a calculated $x=12.7$ inches. Having these two trial values, the designer can obtain the exact value of x by plotting the difference between the assumed and calculated values against the assumed x as abscissas. Such a plot indicates that $x=12.6$ inches,

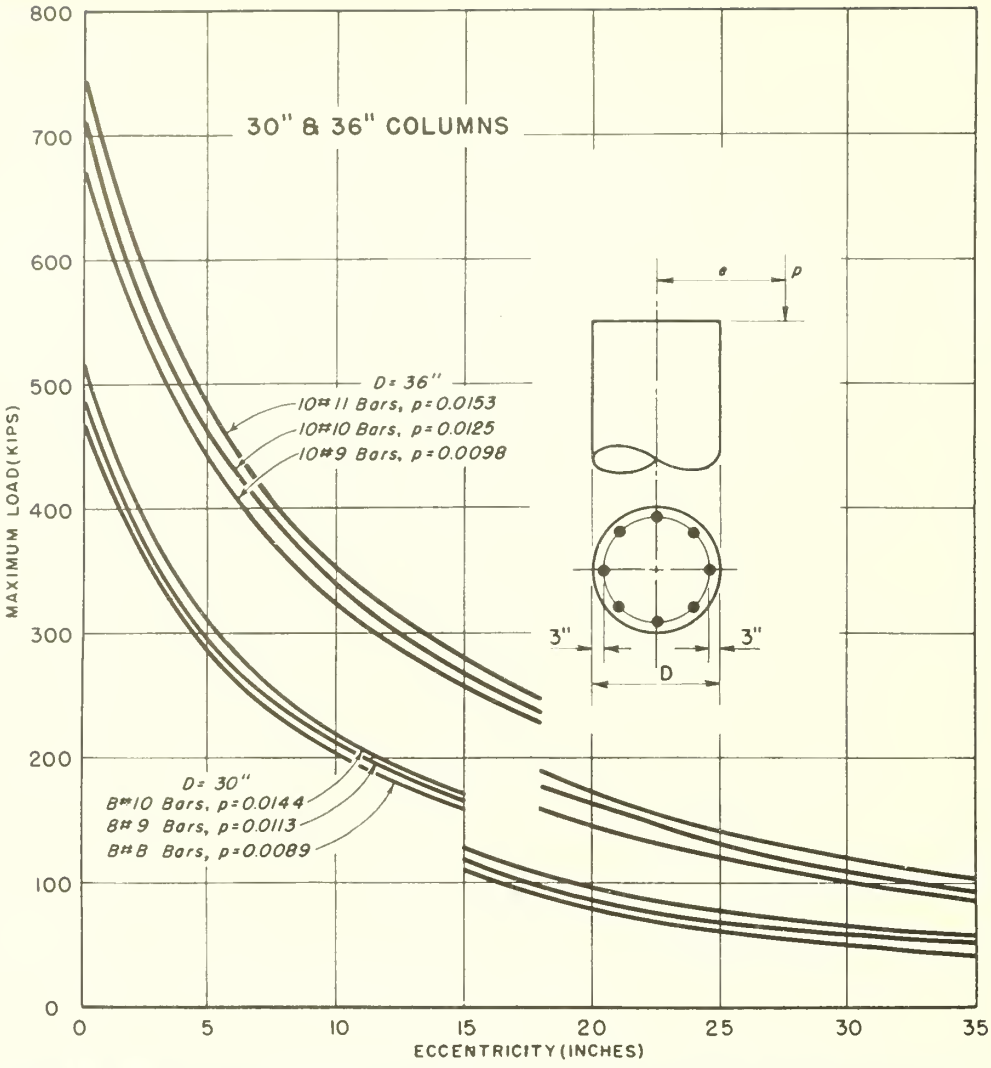


Figure 4.—Maximum load for 30- and 36-inch circular-tied columns in direct stress and bending (f'_c=3,000 p.s.i.; n=10; intermediate, hard and rail steel grade reinforcement).

which when used in the third trial, checks out.

After the value of x has been determined, the C_1 and K quantities and f_a=628 p.s.i. (see example I) are used to obtain the allowable compressive stress. Then,

$$f_e = f_a\left(\frac{1+K(e/t)}{1+C_1K(e/t)}\right)$$--AASHO spec. 1.7.8(f)

$$=628\left(\frac{1+11.2\times0.833}{1+0.523\times11.2\times0.833}\right)$$

$$=628\times\frac{10.33}{5.88}=1{,}105 \text{ p.s.i.}$$

A check for the stress of the compression steel shows that it is critical; and if f_e=1,105 p.s.i. is used, $f'_s/2n$ will exceed the allowable value of 800 p.s.i. ($f'_s=2n\times$ stress in equivalent concrete $\leqq$16,000). If the value of 800 p.s.i. is used, then f_e is reduced to 800×(12.6/9.6)=1,050 p.s.i. The stress in the tension steel becomes 17,000 p.s.i., and in the compression steel, 16,000 p.s.i.

For any column cross section, the properties of the cracked section (I_t, c, $\bar{x}$, etc.) are constant for a given eccentricity, e, and are i dependent of the load, N. The load therefo is directly proportional to stresses. Cons quently, the allowable load is, N=100,000 f_e/f_c=100,000×(1,050/963)=109,000 pound

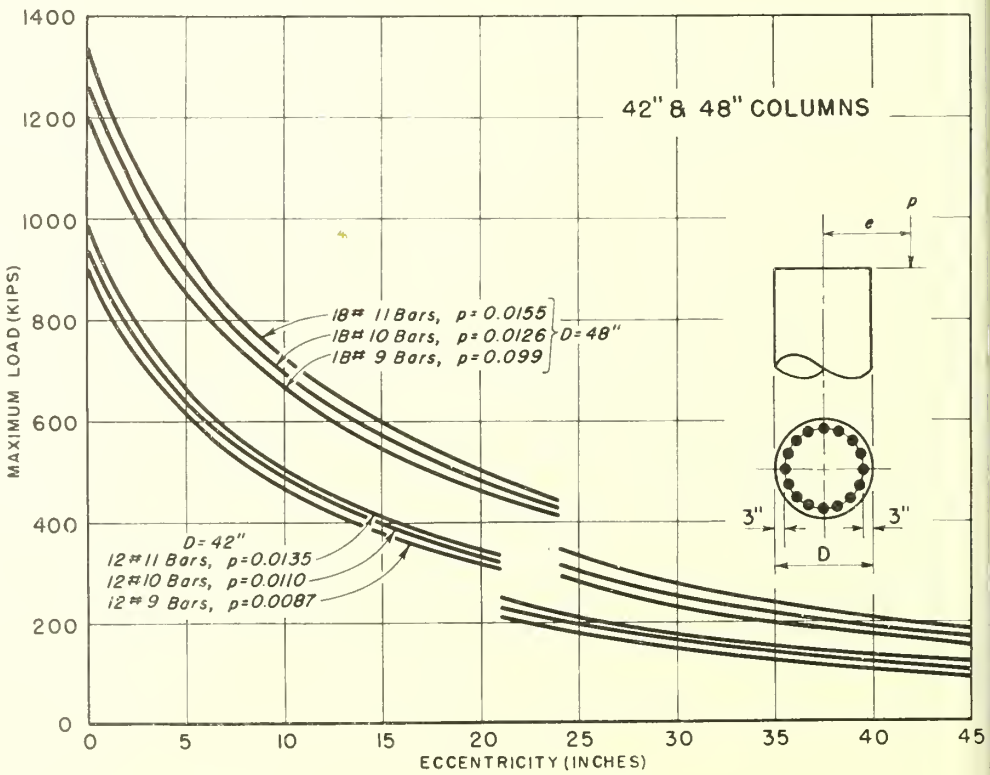


Figure 5 (right).—Maximum load for 42- and 48-inch circular-tied columns in direct stress and bending (f'_c=3,000 p.s.i.; n=10; intermediate, hard and rail steel grade reinforcement).

Nomenclature

A=coefficient (see fig. 1).
A_c=compression area of a column, $A_c=Bt^2$.
A_g=gross cross-sectional area, square inche
A_s=cross-sectional area of longitudinal stee
$A'_s=\lambda A_s$=additional compression steel (s definition of λ).
A_t=total transformed area.
$a=x-t'$.
B=coeffieient (see fig. 1).
C=compressive force.
$C_1=f_a/0.40f'_c$.
c=concrete extreme fiber distance fro bending neutral axis.
D=coefficient (see fig. 1).
d=depth from extreme compression concret fiber to extreme tensile steel fiber.
e=eccentricity measured from center column.
e'=eeeentricity measured from neutral ax for bending of transformed area.
$f_a=P_t/A_t$, stress in concrete of uncracke section.
f_c=computed compressive concrete stres when N is assumed to be 100,000 pounds
f'_c=cylinder strength at 28 days, p.s.i.
f_e=maximum allowable compressive stress i columns subjected to combined axial an bending stress, p.s.i.
f_s=working stress for longitudinal stee nominal or computed.
$I_c=DR^4$= moment of inertia of compressio eoncrete around diameter parallel t neutral axis.
I_s=moment of inertia of steel ring aroun diameter.

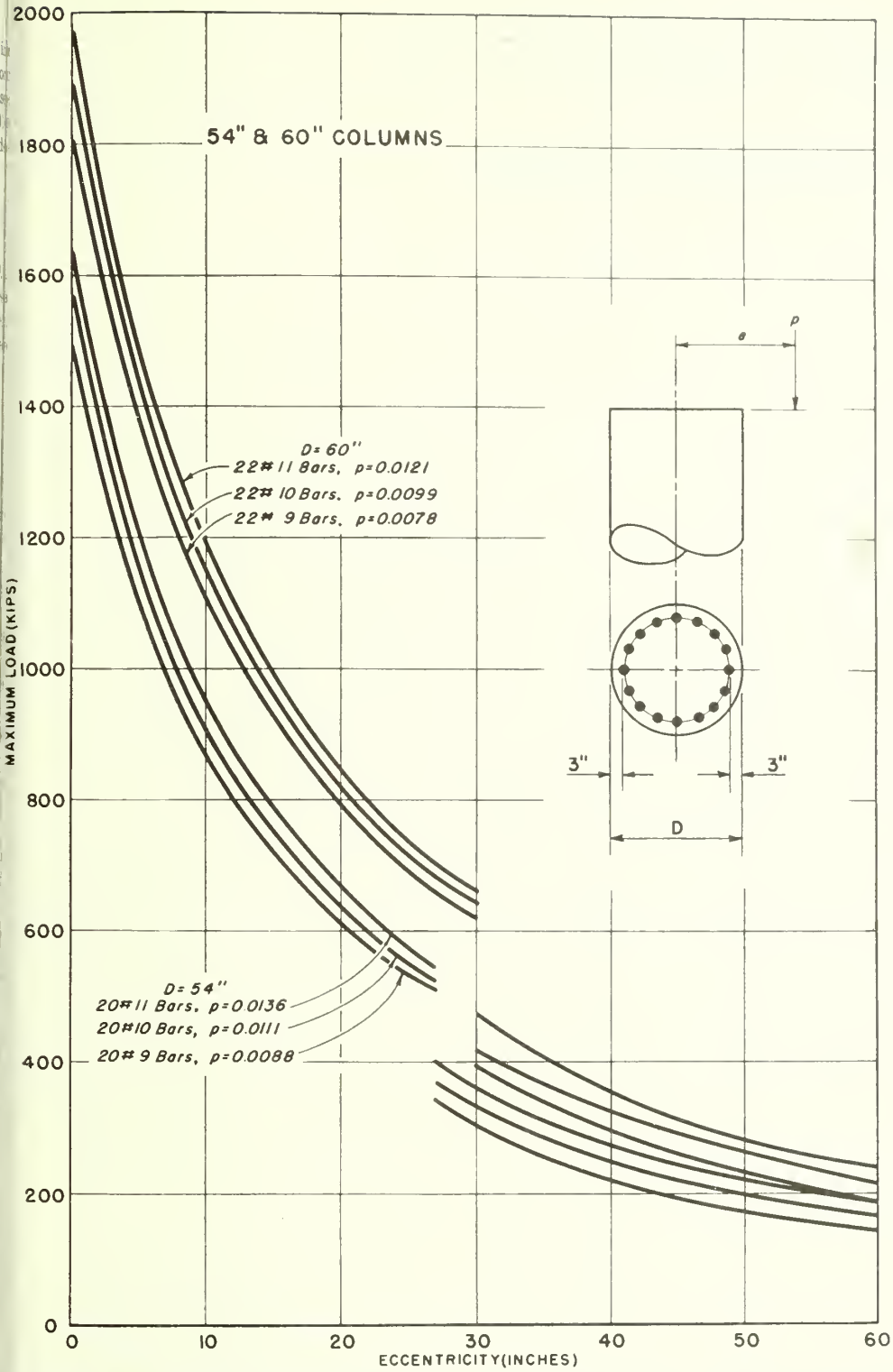


Figure 6.—Maximum load for 54- and 60-inch circular-tied columns in direct stress and bending (f'_c=3,000 p.s.i.; n=10; intermediate, hard and rail steel grade reinforcement).

I_t = total moment of inertia around neutral axis for bending.

$K=t^2/2r^2$, a factor used in the design of eccentrically loaded columns.

$k=x/t$, ratio of depth of compression area to the diameter of column.

M = bending moment.

N = eccentric load.

n = ratio of modulii of steel to concrete.

P_t = total load on tied column centrally applied, pounds.

p = ratio of longitudinal steel area to gross column area.

R = radius of column = $t/2$.

R' = radius of steel ring = $R-t'$.

r = radius of gyration of transformed section.

t = diameter of column.

t' = amount of concrete cover for longitudinal reinforcement.

$x=kt$, depth of compression area.

$x_c=AR$ = distance from center of column to c.g. of compression concrete.

$\bar{x}$ = distance from center of column to neutral axis of transformed area considering bending only.

x' = center of gravity distance of A'_s from centerline of column.

$\lambda=\phi/180=A'_s/A_s$ = ratio of additional compression steel to total area of ring where $\cos\phi=1-a/R'$ and $a=x-t'$.

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Sampling Techniques Applicable to the Collection of Economic Data

BY THE HIGHWAY NEEDS AND ECONOMY DIVISION
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Sample survey techniques are used in many data-collecting phases of highway research. However, not all who must practice sampling have been trained in modern sampling theory. This article presents an account of the theory and its application which is directed primarily to those engaged in sampling for economic data.

Included is a discussion of the basic concepts of sampling, and the advantages and disadvantages of several alternative sampling methods. Proportional stratified sampling and optimum allocation among strata are discussed. Criteria are offered for choosing between these two alternatives with some consideration given to cost factors.

The author suggests one possible application of the theory and offers an illustration based on hypothetical data.

ADMINISTRATORS require sufficient and accurate data upon which to base decisions on broad general policies, as well as on day-to-day problems. Decisions must be made whether pertinent data are available or not. Therefore, data collection and data analyses are the ever-present concern of administrators.

Solution of the problem of data collection is made more difficult by several limiting factors. Two of these are lack of time and lack of money. Sampling is a tool to minimize, to some extent, the effect of these limiting factors. Because sampling cannot effect a limitless spread of resources, administrators must decide on priorities among the data to be collected and on the amount of money to be spent in collection and analysis.

Modern statistical theory not only lights the way to good sample design as an aid to data collection, but it also emphasizes techniques for drawing logical inferences from the the data as an aid to analysis. This latter aspect of statistical theory is not sufficiently emphasized or practiced.

Sampling is not just a stop-gap or substitute for the collection of all data relating to a particular study. It is often not realized that estimates based on relatively small samples may be more reliable than results of complete censuses. Since fewer personnel are needed for sample studies than for complete censuses, there is opportunity for more thorough training and for better supervisory control of employees than is possible in complete censuses. Because data to be processed in a limited time are not amassed for sample studies in the same huge quantities as for complete censuses, an opportunity is presented for better control of the various data processing operations to attain better quality of output. The stress is laid on the word "opportunity." Sampling does not assure quality; it makes improved quality possible with the funds available.

All decisions based on the data collected by the State highway departments and by the Bureau of Public Roads are supported, in one sense, by sample studies. Even complete censuses may be considered sample studies; they are samples in time. Administrators must assume a certain degree of stability or variability over time when making decisions based on such studies. Therefore, anyone who decides to use a complete count is not voting against sampling, but rather against a sample of a given size. In other words, the individual believes that the estimates yielded by the smaller sample will not be sufficiently reliable for the decisions that must emanate therefrom.

Distribution of Sample Estimates

Statistical theory is based on probability samples only. Probability samples are those in which the chance of any element being selected can be calculated before selection and can be expressed as a mathematical ratio. When this condition is satisfied, the probability calculus can be applied to the sample data to make possible meaningful deductions on the reliability of the results, and to fashion improved designs for future sample studies. Use of the probability calculus is impossible with non-probability samples.

Samples yield estimates of the true values of the characteristics of a population. In this context, the word "population" represents the totality of all elements from which a sample may be drawn; the true value is obtained from a study of all elements of a population, not a sample. The word "true" does not imply that the value is fixed even for a point in time. The methods used in determining a true value also influence its magnitude. For example, interviewers possess a given competency and training. The designs of questionnaires and other forms have varying efficiencies in extracting data of high quality. Supervisors and data processors have their own degrees of competency. A change in any of these components will likely result in a different true value.

Sample data collected and processed under a given set of conditions will yield an estimate of the true value that would have been obtained from all elements under the same set of operating conditions. The sample estimate will differ from the true value by an amount of unknown magnitude. Another sample of the same size as the first will yield another estimate of the true value. Usually the estimate yielded by the second sample will differ from the one yielded by the first sample.

Standard deviation

Consider the estimates from all possible samples of the same size drawn from a specified population. These estimates from all possible samples of the same size form another population of their own—a population of sample estimates. They will have a distribution from low estimate to high estimate. As a general rule, the estimates will tend to cluster about the true value. The measure of the scatter of the population of sample estimates about their average value, generally the true value, is called the standard deviation of the population. The standard deviation of the derived population is a function of the standard deviation of the original population.

Normal distribution

The distribution of the population of sample estimates tends toward the normal. The graph of the mathematical function describing the normal distribution exhibits a symmetrical, bell-shaped curve. The area under this curve is distributed in a fixed fashion, whatever the values assigned to the constants in the mathematical function. Distributions of many types of sample estimates approximate the normal with increasing sample size. The larger the sample size, the more closely will the derived population of sample estimates approach the normal distribution. Furthermore, the larger the sample size, the more closely will the sample estimates cluster around the true value, and consequently the smaller will be the standard deviation. The normal distribution is a very satisfactory approximation of the distribution of estimates derived from samples of the sizes ordinarily used by highway departments for economic studies.

Hypothetical population

Some of the remarks on the distribution of sample estimates may be clarified by an illustration based on a hypothetical population of six households which compose a hypothetical State. Assume that residents of five of these households own passenger cars and travel the following annual mileages:

Household number	*Annual mileages*
1	0
2	2,000
3	6,000
4	7,000
5	12,000
6	18,000
Total travel	45,000

Further assume that it is desired to select a simple random sample of these households in order to estimate the total vehicle-miles for the State, and that the estimate is obtained by multiplying the sample values by the reciprocal of the sampling fraction.

The distribution of estimates from all possible samples, as well as the value of the standard deviation and the square of the standard deviation (the variance) for each sample size, is reported in table 1. The formula for calculating the variance, σ^2, is,

$$\sigma^2=\frac{\sum_i^N X_i^2}{N}-\frac{\left(\sum_i^N X_i\right)^2}{N^2}$$

Where:

N=The total number of possible samples.

X_i=The estimate obtained from the i^{th} sample.

The average values for travel shown immediately below the column totals of table 1 illustrate what is meant by an unbiased estimate. A technique yields an unbiased estimate when the average value of all possible estimates of the same item obtained by the application of the technique equals the population value that is being estimated. In this example, the population value is 45,000 vehicle-miles of travel.

A frequency distribution of the estimates in table 1 is presented in table 2 to emphasize the tendency of sample estimates to cluster in a fairly symmetrical fashion about a central value with increasing sample size.

Table 1.—All possible sample estimates of total travel and all corresponding standard deviations derived from a hypothetical population of 6 households

Sample number	Estimated travel (in thousands of vehicle-miles), based on all possible samples of—					
	1 household	2 households	3 households	4 households	5 households	All households
1	0	6	16	22.5	32.4	45
2	12	18	18	30.0	39.6	
3	36	21	26	31.5	45.6	
4	42	24	28	37.5	46.8	
5	72	27	30	39.0	51.6	
6	108	36	36	40.5	54.0	
7		39	38	40.5		
8		42	40	46.5		
9		54	40	48.0		
10		54	42	49.5		
11		57	48	54.0		
12		60	50	55.5		
13		72	50	57.0		
14		75	52	58.5		
15		90	54	64.5		
16			60			
17			62			
18			64			
19			72			
20			74			
Total, all samples	270	675	900	675	270	45
Average per sample	45	45	45	45	45	45
Variance	1,317.0	526.8	263.4	131.7	52.68	0
Standard deviation	36.3	23.0	16.2	11.5	7.3	0

Standard Error of Estimate

Ordinarily, the results of only one sample are available, not of all possible samples. From this one probability sample can be calculated not only an estimate of the true value, but also an estimate of the standard deviation of the population of sample true values. The latter estimate is called the standard error of estimate. The larger the sample size for a given design, the smaller will be the standard error of estimate.

Using the sample estimate of the true value and the standard error of estimate calculated from the sample, a range of values can be determined within which the unknown true value probably lies. This statement may or may not be correct in a given situation. Because the distribution of estimates from large samples tends to approximate the normal, it can be shown that, were similar statements made for all possible samples of the same size, using the individual sample estimates each time, approximately 67 percent of the statements would be correct, provided that the range were based on one standard error of estimate. Approximately 95 percent of the statements would be correct, provided the range were based on two standard errors of estimate. In any given case, it may be assumed that the sample is one of 67 out of 100 (one standard error) that yields ranges which include the true value; or, that it is one of 95 out of 100 that yields such ranges based on two standard errors. The confidence levels based on the results of a single sample can therefore be controlled by the number of standard errors that are used in defining the range. Moreover, for a given confidence level, the larger the sample size, the narrower is the range within which the true value probably lies.

Within practical limits, the more important the decision to be based on the results of a sample, the more money will be paid to reduce the range, determined for a given confidence level, within which the true value lies. In other words, the more important the use to which the estimate will be put, the greater is the precision desired in the estimate.

Some of the characteristics of standard errors of estimate and of their squares, the variances, can be illustrated with the hypothetical population of six households. The values for the variances and standard deviations, shown in table 3 and illustrated in figure 1, were derived from the formula,

$$s^2=\left[N^2\frac{N-n}{Nn}\right]\left[n\sum_i^n x_i^2-\left(\sum_i^n x_i\right)^2\right]\div\frac{1}{n(n-1)}$$

Where:

N=The population (six households).

n=The sample size.

x_i=The value of the i^{th} household in a sample.

The following observations may be made concerning table 3: (1) A sample containing only one unit cannot yield an estimate of the spread of the population of estimates from all possible samples which contain only one unit. (2) The formula results in an unbiased estimate of the variance as shown by comparing the values on the last line of table 3 with the next to the last line of table 1. (3) Similarly, the standard error of estimate is not an unbiased estimate of the standard deviation of the population of estimates from all possible samples of a given size. The bias decreases with increases in sample sizes. (The sample sizes ordinarily employed by the State highway departments in planning studies should result in standard errors with negligible bias.) (4) The larger the sample size, the more stable

Table 2.—Frequency distribution of all possible sample estimates of total travel derived from a hypothetical population of 6 households

Class limits of estimated travel (1,000 vehicle-miles)	Number of samples having indicated range of travel, based on all possible samples of—					
	1 household	2 households	3 households	4 households	5 households	All households
0– 9.9	1	1				
10–19.9	1	1	2			
20–29.9		3	2	1		
30–39.9	1	2	3	4	2	
40–49.9	1	1	4	5	2	1
50–59.9		3	4	4	2	
60–69.9		1	3	1		
70–79.9	1	2	2			
80–89.9						
90–99.9		1				
100–109.9	1					

Table 3.—Standard errors of estimates, s, and their squares, s^2, for each of all possible estimates of travel derived from a hypothetical population of 6 households

Sample number	Standard errors and their squares for estimates of travel, based on all possible samples of—											
	1 household		2 households		3 households		4 households		5 households		All households	
	s^2	s	s^2	s	s^2	s	s^2	s	s^2	s	s^2	s
1	(1)	(1)	24	4.9	56	7.5	32.75	5.7	26.16	5.1	0	0
2	(1)	(1)	216	14.7	78	8.8	84.00	9.2	58.56	7.7		
3	(1)	(1)	294	17.1	86	9.3	86.75	9.3	65.76	8.1		
4	(1)	(1)	96	9.8	248	15.7	72.75	8.5	65.04	8.1		
5	(1)	(1)	150	12.2	42	6.5	195.00	14.0	54.96	7.4		
6	(1)	(1)	864	29.4	216	14.7	194.75	14.0	45.60	6.8		
7			6	2.4	218	14.8	50.75	7.1				
8			600	24.5	584	24.2	168.75	13.0				
9			1,944	44.1	152	12.3	216.00	14.7				
10			216	14.7	150	12.2	140.75	11.9				
11			150	12.2	504	22.4	180.00	13.4				
12			1,536	39.2	494	22.2	174.75	13.2				
13			864	29.4	62	7.9	147.00	12.1				
14			726	26.9	416	20.4	140.75	11.9				
15			216	14.7	402	20.0	90.75	9.5				
16					504	22.4						
17					266	16.3						
18					392	19.8						
19					216	14.7						
20					182	13.5						
Total, all samples			7,902	296.2	5,268	305.6	1,975.50	167.5	316.08	43.2	0	0
Average per sample			526.8	19.7	263.4	15.3	131.70	11.2	52.68	7.2	0	0

[1] Not applicable.

is the value of the standard error from sample to sample.

The length of each horizontal line in figure 1 represents twice the standard error given in table 3 for each of the sample estimates. The starting point, indicated by the fork at one end of each horizontal line, is the estimate of total travel shown in table 1 for each sample of a given sample size. Each horizontal line is extended in the direction of the true value of total travel, the heavy vertical line at the scale value of 45. If the sample estimate is less than 45, the value of 2 standard errors is added; and if the sample estimate is greater than 45, the value of 2 standard errors is subtracted. The spacing between horizontal lines has no significance.

Figure 1 shows that the various sample estimates of total travel (table 1) plus or minus twice the corresponding standard errors (table 3) produce ranges of values which include the population total that is being estimated in the following proportion of cases: sample size 2, 11/15; sample size 3, 15/20; sample size 4, 13/15; and sample size 5, 5/6. These proportions are poor approximations of the proportion of 95 in 100 which was previously mentioned. It must be remembered that the illustration is based on a small population with small sample sizes.

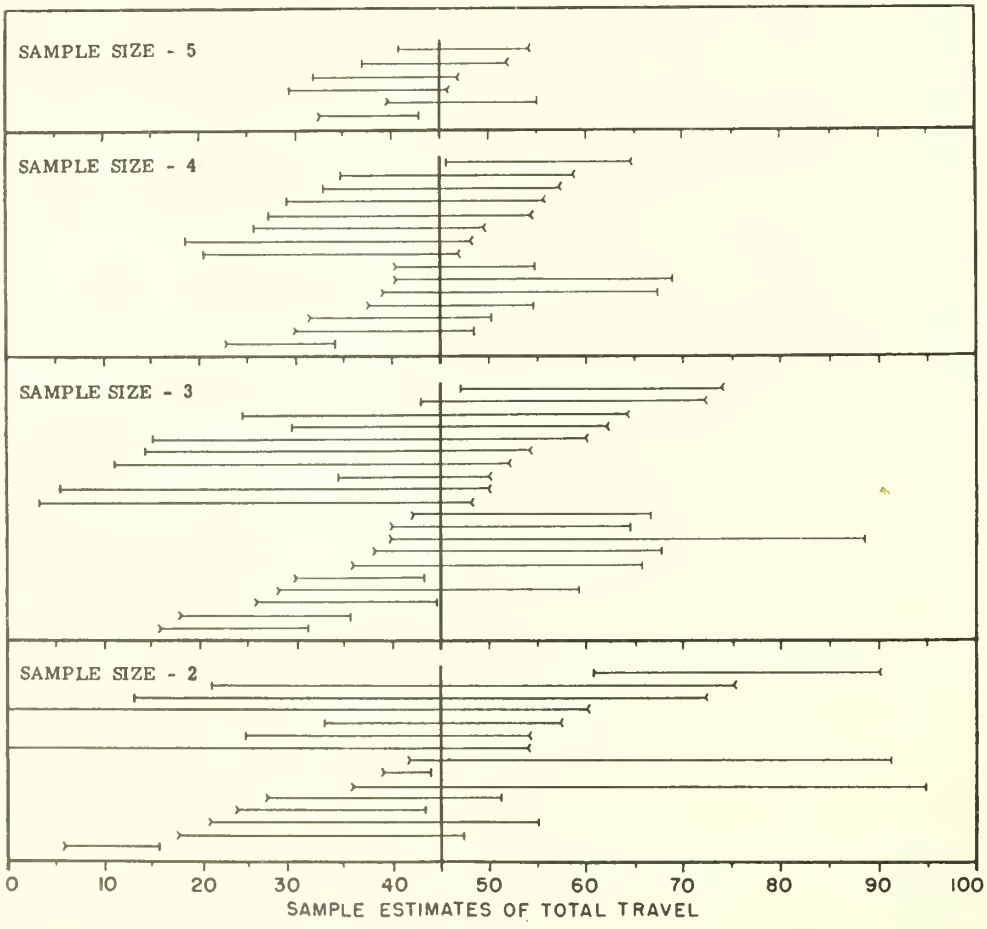

Figure 1.—All possible samples of a given sample size, plus or minus twice the corresponding standard errors (45 is the population value being estimated).

Optimum Sample Design

An important area in modern sampling theory has been the development of criteria for designing samples that will yield optimum results. The optimum is defined either as a specified standard error for the least cost, or as the smallest standard error for a given cost. Two sets of estimates and a predetermined standard are required in order to make use of the theory. The required sets are estimates of the unit costs of various study operations and the standard deviations of the several groupings into which will be allotted all the elements making up the original population. The organization paying for the study must predetermine either the total amount of money to be spent on a study, or the standard error to be attained for a given sample estimate. Often the latter is specified and an upper limit is placed on the money to be spent.

Estimates of the standard deviations of the several groupings must be calculated from data obtained in previous samples, from small-scale pilot studies, or from more or less educated guesses. Estimates of unit costs can be based on past experience.

Insufficient data on variances and costs have made it difficult to ascertain whether an optimum sample design has been attained in past studies in which the Bureau of Public Roads has been interested. However, efforts have been made to achieve a specified standard error. For example, in both the latest motor-

vehicle-use studies and motor-vehicle accident cost studies, the samples were designed to attain standard errors that would be about 7 percent of the estimate for a given total at the level of one standard error.

One complication in the consideration of optimum design is that economic and traffic studies have been multi-purpose studies, and will likely continue to be multi-purpose studies. It is impossible to design one sample survey to produce a multitude of estimates, each with its own prespecified standard error. Someone must determine which single estimate, or possibly two or three estimates, are the most important. The sample is designed to attain the specified standard error or standard errors for these important estimates. The standard errors of the other subsidiary estimates must be accepted at whatever value the sample yields. For example, in the motor-vehicle-use studies the aim is now to attain the specified standard errors for two estimates: the total vehicle-miles for passenger cars driven by residents of a State, and the total vehicle-miles for trucks registered in a State. In the accident cost studies for passenger cars, the aim is now to attain the specified standard errors for three estimates: direct costs of fatal accidents, injury accidents, and property-damage-only accidents.

Considerations in Sample Design

Basic elements

Thus far, some of the broad aspects of sample design have been viewed from afar. At this point some of the details of the problem of sample design will be considered at close range. This consideration begins with two definitions: An *elementary unit* is an individual member of the particular population on which measurements are desired and for which analyses are to be made from the survey results. A *sampling unit* is either one elementary unit or a number of elementary units which are grouped and treated as one for sampling purposes.

Equating precision and cost

Determination of the optimum sampling unit is not always simple. For example, a city may be regarded as a number of city blocks, a number of households, or a number of persons. The optimum unit is one which gives the desired precision for the sample estimate at the smallest cost, or the greatest precision for a fixed cost. For a given sample size, a large sampling unit is usually less expensive than a small unit, but it is often less precise. To attain a specified precision in the overall sample, it is often necessary to get data from more elementary units when they are grouped into sampling units than if the elementary units are sampled directly. The choice of sampling unit involves striking a balance between relative precision and relative cost. Elementary units are grouped most often into sampling units when the sample design calls for one or more stages of subsampling.

Two types of grouping of sampling units are practiced: units grouped into strata and units grouped into clusters. Each type is formed under different conditions, but the underlying sampling principle is to achieve maximum precision per unit of cost. Both types of groupings may be practiced in the same sample design; use of one does not eliminate the other.

Stratified samples

A stratified sample is one in which the elements of the population are separated into fairly homogeneous groups and a sample is drawn to represent each group, each sample being independent of the one drawn from any other group. Stratification is resorted to in the hope of attaining increased precision for a given sample size, or a specified precision for a smaller sample size. The amount of increase in precision of sample estimates accomplished by stratification will depend on the degree of homogeneity that is achieved within strata. This, in turn, depends on how effectively the strata have been defined.

In defining strata, use should be made of all information that helps classify members of the population into groups which differ from one another with respect to the characteristic being measured, or with respect to the cost of collecting data. But, within each stratum, the sample must be a probability sample.

Unless relatively large differences can be set up among the strata, the gain in precision will be only moderate or perhaps barely noticeable. Stratification is sometimes adopted for administrative convenience and money savings with no prospect of reducing standard errors. This often occurs when a stable field organization exists, and it is less costly to use the established setup than to form a temporary organization for a single effort.

The criteria used in forming strata should be fairly well correlated with the changes in the value of the important characteristic so that stratum boundaries may be readily determined. As a general rule, most of the gain in precision has been attained with the use of one criterion that is strongly correlated with the values of the item to be estimated. Other criteria that are well correlated with the values of the item to be estimated are probably also well correlated with the first criterion. For two reasons, therefore, it is often uneconomical to spend much time on stratification. In the first place, stratification may provide little gain in precision since no marked differences can be segregated; and secondly, most of the gain has been realized in the application of one or two well-correlated criteria.

The preceding discussion should not lead anyone to forget the obverse. Stratification can yield marked gains in precision, the amount of the gain being proportional to sizes of the differences among the strata.

A general rule for optimum results is that the number of sampling units to be selected from any stratum should be directly proportional to the variability of the units and inversely proportional to the square root of the costs for collecting and processing the data from the stratum.

Cluster samples

The formation of clusters for sampling purposes is most often resorted to in area sampling. Clusters are formed from the sampling units or elementary units that are found in a more or less compact area. Cluster sampling may also be applied to lists or to files of reports as a technique to save examination of every unit or to save listing costs.

The guiding principle in the formation of clusters is that the component units be as heterogeneous as possible. This is the opposite of the rule for forming strata. Insofar as possible, each cluster is made as heterogeneous as the population, or as some subpopulation. If this were perfectly possible, the examination of all elements in one cluster would be sufficient to yield an acceptable estimate. This is generally not possible. Contiguous units tend to have similar characteristics. Because of this tendency, a cluster sample of a given size yields less precise estimates than stratified samples of the same size. Hence, somewhat larger sample sizes are required for cluster samples than for stratified samples to attain a desired precision of estimate. The increase in sample size is a function of the average similarity of the units composing the clusters.

Despite the increase in sample size, cluster sampling often results in reduced costs, and the cost reduction may be substantially greater than that experienced with simple random sampling of ungrouped data or stratified sampling. These cost economies are realized in savings in travel costs, administrative costs, and listing costs.

Stratification and cluster sampling may be used in the same sample survey. They are merely methods of grouping. Stratification can yield a gain in precision for a given sample size or result in a smaller sample for a given precision. Clustering can yield a savings in certain aspects of survey costs despite an increase in sample size.

Selecting the Sampling Method

Whether sampling units are grouped or not, they must still be selected for a study. Three selection methods are in common use: simple random sampling, systematic sampling, and sampling with unequal probabilities. Since the first two methods are often treated synonymously, and since the third may be unfamiliar, it may be well to define these terms.

A *simple random sampling* of n sampling units is one selected so that each combination of the n units has the same chance of being selected as any other combination.

A *systematic sample* of n sampling units is one in which a constant selection interval is applied n times in step fashion to the ordered elements of a population.

A *sample selected with unequal probabilities* is one in which the sampling units composing a population have been assigned varying weights, and the chance for the selection of any unit is proportional to its assigned weight.

The basic difference between systematic and simple random samples is that not every com-

(*Continued on page 252*)

STATE LEGAL MAXIMUM LIMITS OF MOTOR VEHI

Prepared by t

Line	State	Width inches[1]	Height ft.-in.	Length—feet[2]				Number of towed units[3]			Axle load—pounds				Type of restri
				Single unit		Truck tractor semi-trailer	Other combination	Semi-trailer	Full trailer	Semi-trailer and full trailer	Single		Tandem		
				Truck	Bus						Statutory limit	Including statutory enforcement tolerance	Statutory limit	Including statutory enforcement tolerance	
1	Alabama	96	[6] 12-6	35	40	50	NP	1	NP	NP	18,000	19,800	36,000	39,600	Table
2	Arizona	96	13-6	40	40	65	65	1	1	2	18,000		32,000		Table
3	Arkansas	96	13-6	35	40	50	50	1	1	NP	18,000	[7] 18,500	32,000	32,500	Spec. maximum[8]
4	California	96	13-6	35	[9] 35	[10] 65	[10] 65	NR	NR	NR	18,000		32,000		Table
5	Colorado	[11] 96	[12] 13-6	35	40	60	60	1	2	[13]	18,000		36,000		Formula-spec. l
6	Connecticut	102	12-6	50	50	50	NP	1	NP	NP	22,400	22,848	36,000	36,720	Spec. lim.-tire c
7	Delaware	96	[6] 12-6	40	42	50	60	1	1	2	20,000		36,000		Table-spec. lim
8	District of Columbia	96	12-6	35	35	50	80	1	1	NP	22,000		38,000		Table
9	Florida	96	[6] 12-6	[14] 35	40	50	50	1	1	NP	20,000	22,000	40,000	44,000	Table
10	Georgia	96	13-6	[15] +39	[15] +45	50	50	1	1	NP	18,000	20,340	36,000	40,680	Spec. maximum[16]
11	Hawaii	108	13-0	40	40	55	65	1	1	2	24,000		32,000		Formula[17]
12	Idaho	[18] 96	14-0	35	[19] 40	60	65	1	1	2	[20] 18,000		[20] 32,000		Table[20]
13	Illinois	96	13-6	42	42	50	50	1	1	2	[21] 18,000		32,000		Spec. lim.-tire c
14	Indiana	96	13-6	36	40	50	50	1	1	2	[23] 18,000	[23] 19,000	[23] 32,000	[23] 33,000	Spec. lim.-tire c
15	Iowa	96	13-6	35	[19] 40	50	NP	1	[24] 1	NP	18,000	18,540	32,000	32,960	Table
16	Kansas	96	13-6	35	[19] 40	50	50	1	1	NP	18,000		32,000		Table
17	Kentucky	96	[25] 13-6	[26] 35	[26] 35	[27] 50	NP	1	NP	NP	18,000	[28] 18,900	32,000	[28] 33,600	Spec. lim.-tire c
18	Louisiana	96	[6] 12-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Axle lim.-tire c
19	Maine	96	[30] 12-6	50	50	50	50	1	1	NP	[30] 22,000		[30] 32,000		Table-tire cap.
20	Maryland	96	[6, 45] 12-6	55	55	55	55	NR	NR	NR	22,400		[31] 40,000		Formula
21	Massachusetts	96	NR	35	[19] 40	[31] 50	NP	1	NP	NP	22,400		36,000		Table-spec. li
22	Michigan	96	13-6	35	40	55	55	1	1	2	[33] 18,000		[34] 32,000		Axle lim.-tire
23	Minnesota	96	13-6	40	40	50	50	1	1	NP	18,000		32,000		Table
24	Mississippi	96	[6] 12-6	35	40	[36] 50	50	1	1	NP	18,000		28,650	[35] 32,000	Table-tire cap
25	Missouri	96	12-6	35	40	50	50	1	1	2	18,000		32,000		Table
26	Montana	[18] 96	13-6	35	40	60	60	1	1	[37] 2	18,000		32,000		Table
27	Nebraska	96	13-6	40	40	60	60	1	1	2	18,000	18,900	32,000	33,600	Table
28	Nevada	96	NR	NR	NR	NR	NR	NR	NR	NR	18,000	18,900	32,000	33,600	Table
29	New Hampshire	96	13-6	35	[35] 40	50	50	NR	NR	NR	22,400		36,000		Tables-spec. l
30	New Jersey	96	13-6	35	[39] 35	45	[40] 50	1	1	NP	22,400	23,520	32,000	33,600	Spec. limits
31	New Mexico	[41] 96	13-6	40	40	65	65	1	1	2	21,600		34,320		Table
32	New York	96	13-0	35	[42] 35	50	50	1	1	NP	22,400		36,000		Formula
33	North Carolina	96	[6] 12-6	35	[19] 40	[43] 50	[43] 50	1	1	NP	18,000	19,000	36,000	38,000	Spec. limits
34	North Dakota	96	13-6	[14] 35	[19] 40	60	60	1	1	2	18,000		32,000		Formula
35	Ohio	96	13-6	35	[19] 40	50	60	1	NR	NR	19,000		[44] 41,500		Formula
36	Oklahoma	96	13-6	35	45	50	50	1	1	NP	18,000		32,000		Table
37	Oregon	96	[45] 12-6	35	[35] 40	[46, 35] 55	[35] 65	1	1	[35] 2	[47] 18,000		[47] 32,000		Table[48]
38	Pennsylvania	96	[6] 12-6	35	[19] 40	50	[40] 50	1	1	NP	22,400	23,072	36,000	37,080	Spec. limits[49]
39	Puerto Rico	96	12-6	N5	N5	N5		N5	N5	N5	N5		N5		Spec. lim.-tire
40	Rhode Island	102	12-6	40	40	50	50	1	1	NP	22,400		N5		Spec. limits
41	South Carolina	96	[45] 12-6	[14] 35	[19] 40	50	60	1	1	NP	20,000		32,000		Table
42	South Dakota	96	13-0	35	40	50	60	1	1	2	18,000		32,000		Table
43	Tennessee	96	[6] 12-6	35	40	50	50	1	[53] 1	NP	18,000		32,000		Table
44	Texas	96	13-6	35	40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table[54]
45	Utah	96	14-0	45	45	60	60	NR	NR	NR	18,000		33,000		Table
46	Vermont	96	12-6	50	50	50	50	1	1	NP	N5		N5		Spec. lim.-tire
47	Virginia	96	[6] 12-6	35	[35] 40	50	50	1	1	NP	18,000		[57] 32,000		Table
48	Washington	96	13-6	35	[19] 40	60	[58] 65	1	1	[58] 2	18,000	[59] 18,500	32,000	[59] 33,000	Table-spec. lir
49	West Virginia	96	[6] 12-6	35	[19] 40	50	50	1	1	NP	18,000	18,900	32,000	33,600	Table
50	Wisconsin	96	[6] 12-6	35	40	50	50	1	1	NP	18,000	[60] 19,500	30,000	32,000	Table-formula[61]
51	Wyoming	96	13-6	40	40	60	60	1	1	2	18,000		32,000	[62] 36,000	Table
52	Alaska	96	12-6	35	[19] 40	60	60	1	1	2	18,000		32,000		Table-tire cap
	AASHO Policy	96	12-6	35	[19] 40	50	60	1	1	NP	18,000		32,000		Table
Number of States	Higher	3	30	18	29	18	10	6	8	25	31		29		Formula 6
	Same	49	22	34	19	33	12	46	40	27	21		21		Table 12
	Lower	0	0	0	4	1	30	0	4	0	0		2		Specified limi 4

NP—Not permitted. NR—Not restricted. N5—Not specified.

[1] Various exceptions for farm and construction equipment; public utility vehicles; house trailers; urban, suburban, and school buses; haulage of agricultural and forest products; at wheels of vehicles; for safety accessories, on designated highways, and as administratively authorized.

[2] Various exceptions for utility vehicles and loads, house trailers and mobile homes.

[3] When not specified, limited to number possible in practical combinations within permitted length limits; various exceptions for farm tractors, mobile homes, etc.

[4] Legally specified or established by administrative regulation.

[5] Computed under the following conditions to permit comparison on a uniform basis between States with different types of regulation:

A. Front axle load of 8,000 pounds.

B. Maximum practical wheelbase within applicable length limits:

(1) Minimum front overhang of 3 feet.

(2) In the case of a 4-axle truck-tractor semitrailer, rear overhang computed as necessary to distribute the maximum possible uniform load on the maximum permitted length of semitrailer to the single drive-axle of the tractor and to the tandem axles of the semitrailer, within the permitted load limits of each.

(3) In the case of a combination having 5 or more axles, minimum possible combined front and rear overhang assumed to be 5 feet, with maximum practical load on maximum permitted length of semitrailer, subject to control of loading on axle groups and on total wheelbase as applicable.

C. Including statutory enforcement tolerances as applicable.

[6] Auto transports 13 feet 6 inches.

[7] Does not apply to combinations of adjacent load-carrying single axles.

[8] 56,000 pounds on load-carrying axles, exclusive of steering-axle load.

[9] On specific routes in urban or suburban service under special permit from P.U.C. 40 feet, also 3-axle buses with turning radius less than 45 feet without restriction.

[10] Effective September 18, 1959 on Interstate, 4-lane, and designated State highways.

[11] Buses 102 inches.

[12] On designated highways; 12 feet 6 inches on other highways.

[13] Legal limit 60,000 pounds, axle spacing 27 feet or more.

[14] Three-axle vehicles 40 feet.

[15] Truck 39.55 feet; bus 45.20 feet.

[16] 63,280 pounds maximum, except on roads under F

[17] 700 (L+40) when L is 18' or less; 800 (L+40) w with span of 20' or over.

[18] Buses 102 inches on highways of surfaced width

[19] Less than three axles 35 feet.

[20] Special limits for vehicles hauling timber and tim including livestock; single axle 18,900 pounds, tandam mitted 66,000 pounds maximum at 21-foot axle spacing, foot axle spacing.

[21] On designated highways; 16,000 pounds on other

[22] Without tandam axles 45,000 pounds.

[23] On designated highways; single axle 22,400 poun excesses of weight under one or more limitations of axl

[24] Limited to 4 wheel trailer towed by truck not exce

[25] Class AA highways; 12 feet 6 inches on other hig

[26] On designated highways; trucks 26.5 feet and bus

[27] Class AA highways; 45 feet on other highways.

[28] Class AA highways only.

[29] Maximum gross weight on Class A highways 42,0

[30] Including load 14 feet; various exceptions for vel

[31] Effective September 21, 1959.

[32] Subject to axle and tabular limits.

[33] Single axle spaced less than 9 feet from nearest

[34] On designated highways only and limited to one to

[35] On designated highways only.

[36] Auto transports permitted 50 feet.

[37] Semitrailer and semitrailer converted to full trail

WEIGHTS COMPARED WITH AASHO STANDARDS

ds, July 1, 1959

Applicable to:		Specified maximum gross weight—pounds[4]						Practical maximum gross weight—pounds[5]						Line
		Truck		Truck-tractor semitrailer				Truck		Truck-tractor semitrailer				
Any group of axles	Total wheel base only	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	2-axle	3-axle	3-axle	4-axle	5-axle	Other combination	
	X							27,800	47,600	47,600	60,010	64,650	NP	1
Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,800	2
Under 18'	Over 18'							26,500	40,500	45,000	59,000	65,000	65,000	3
								26,000	40,000	44,000	58,000	72,000	76,000	4
	X	30,000	46,000					26,000	44,000	44,000	62,000	76,000	76,000	5
		32,000	50,000	50,000	60,000	60,000	NP	30,848	44,720	51,000	61,200	61,200	NP	6
X		30,000	46,000	48,000	60,000	60,000	60,000	28,000	48,000	48,000	56,350	60,000	60,000	7
X								30,000	46,000	52,000	58,450	61,490	64,650	8
	X							30,000	52,000	52,000	65,200	73,095	73,095	9
							63,280	28,340	48,680	48,680	63,280	63,280	63,280	10
X								32,000	38,800	56,000	62,800	69,600	78,000	11
X								26,000	40,000	44,000	58,000	72,000	76,800	12
		36,000	[22] 41,000	45,000	59,000	72,000	72,000	26,000	40,000	44,000	58,000	72,000	72,000	13
							72,000	27,000	41,000	45,000	59,000	[23] 73,000	[23] 73,000	14
X								26,540	40,960	45,080	59,500	73,280	NP	15
X								26,000	40,000	44,000	55,470	63,890	63,890	16
		36,000	50,000	54,000	59,640	59,640	NP	26,900	41,600	45,800	59,640	59,640	NP	17
								26,000	40,000	44,000	58,000	72,000	76,000	18
X		32,000	[30] 50,000	50,000	60,000	60,000	60,000	30,000	40,000	52,000	60,000	60,000	60,000	19
	X			65,000	65,000	65,000	65,000	30,400	48,000	52,800	65,000	65,000	65,000	20
	X	[32] 46,000	[32] 60,000	[32] 60,000	[32] 60,000	[32] 60,000	NP	30,400	44,000	52,800	60,000	60,000	NP	21
								26,000	[35] 40,000	44,000	[35] 58,000	[35] 66,000	[35] 102,000	22
X								26,000	40,000	44,000	58,000	68,000	72,500	23
X								26,000	36,650	44,000	54,650	[35] 58,000	[35] 58,000	24
X								26,000	40,000	44,000	55,470	64,650	64,650	25
Under 18'	Over 18'							26,000	40,000	44,000	58,000	72,000	76,000	26
X		36,000	54,000	54,000	71,146	71,146	71,146	26,780	41,200	45,320	59,740	73,280	73,280	27
Under 18'	Over 18'							26,900	41,600	45,800	60,000	74,000	76,800	28
	X	33,400	[38] 47,500	52,800	66,400			30,400	44,000	52,800	66,400	66,400	66,400	29
		30,000	40,000	60,000	60,000	60,000	60,000	31,500	41,600	55,040	63,000	63,000	63,000	30
Under 18'	Over 18'							29,600	42,320	51,200	63,920	76,640	86,400	31
	X					65,000	65,000	30,400	44,000	52,800	65,000	65,000	65,000	32
		31,500	46,200	46,200	65,100	65,100	65,100	27,000	46,000	46,000	65,100	65,100	65,100	33
Under 18'	Over 18'							26,000	38,000	44,000	56,000	64,000	64,000	34
	X							27,000	39,500	46,000	58,500	71,000	78,000	35
X								26,000	40,000	44,000	58,000	72,000	73,280	36
Under 18'	Over 18'					[48] 74,000	[48] 76,000	26,000	40,000	44,000	58,000	[48] 72,000	[48] 76,000	37
		33,000	47,000	50,000	60,000	60,000	62,000	31,072	45,080	51,500	61,800	61,800	63,860	38
														39
		[50] 36,000	[51] 44,000	[52] 50,000	[13] 60,000	60,000	88,000	30,400	44,000	50,000	60,000	60,000	88,000	40
X								28,000	40,000	48,000	60,000	66,839	71,115	41
X								26,000	40,000	44,000	58,000	72,000	73,280	42
	X							26,000	40,000	44,000	58,000	61,580	43,500	43
X								26,900	41,600	45,800	60,500	75,200	75,600	44
X								26,000	41,000	44,000	59,000	74,000	79,900	45
		30,000	[55] 50,000	50,000	[56] 60,000	[56] 60,000	[56] 60,000	30,000	[55] 50,000	50,000	60,000	60,000	60,000	46
X						[35] 56,800	[35] 56,800	26,000	40,000	44,000	56,800	56,800	56,800	47
Under 18'	Over 18'	28,000	36,000	46,000	60,000	68,000	72,000	26,000	36,000	44,000	60,000	68,000	72,000	48
X								26,900	41,600	45,800	57,844	63,840	63,840	49
X								27,500	40,000	47,000	59,500	68,000	68,000	50
X								26,000	44,000	44,000	62,000	73,950	73,950	51
Under 18'	Over 18'	29,000	43,000	47,000	61,000	75,000	75,000	26,000	40,000	44,000	58,000	72,000	75,000	52
X								26,000	40,000	44,000	55,470	61,490	71,900	
20	18							29	27	29	47	40	25	
								22	19	22	3	2	0	
								0	5	0	1	9	26	

000 pounds maximum.
900 (L+40) on highways having no structures
rise as administratively authorized.
ntrates, aggregates, and agricultural products
s weight table: vehicle with 3 or 4 axles per-
xles permitted 79,000 pounds maximum at 43-
ounds; tolerance of 1,000 pounds on total of all
ight.
ways.
ighways 30,000 pounds.
lucts and construction materials.
unds.
n; otherwise 26,000 pounds.

[38] Dual-drive axles; otherwise 40,000 pounds.
[39] Or as prescribed by P.U.C.
[40] Exception for palos, pilings, structural units, etc., permitted 70 feet.
[41] On designated highways 102 inches.
[42] Trackless trolleys and buses 7 passengers or more, P.S.C. certificate 40 feet.
[43] Including front and rear bumpers.
[44] Approved equipment 73,280 pounds.
[45] Certain types of vehicles and commodities under special permit on designated highways up to 13 feet 6 inches.
[46] 60 feet allowed truck tractor semitrailer on major routes designated by permit.
[47] Logging vehicles permitted 3-foot wheelbase tolerance, 19,000-pound single axle, 34,000-pound tandem axle.
[48] Governs gross weight permitted on highways designated by resolution of State highway commission or by permit.
[49] Single unit truck with 4 axles permitted 60,000 pounds.
[50] Axles spaced less than 6 feet 32,000 pounds; less than 12 feet 36,000 pounds; 12 feet or more gross weight governed by axle limit.
[51] Single vehicle with 3 or more axles spaced less than 16 feet 40,000 pounds; less than 20 feet 44,000 pounds; 20 feet or more governed by axle limit.
[52] Tractor semitrailer with 3 or more axles spaced less than 22 feet 46,000 pounds; not less than 27 feet 50,000 pounds.
[53] Limited to 3,500 pounds.
[54] Effective January 1, 1960.
[55] On Interstate routes 40,000 pounds.
[56] Tandem axles on trailer equipped with adequate brakes.
[57] Vehicles registered before July 1, 1956, permitted limits in effect January 1, 1956, for life of vehicle.
[58] Three-unit combinations on designated highways.
[59] Vehicles hauling logs permitted wheelbase and gross weight tolerances. Discretionary enforcement tolerances not included in computation of practical maximum gross weights.
[60] Axle load 21,000 pounds on 2-axle trucks hauling unmanufactured forest products.
[61] On Class A highways.
[62] Based on ruling of Attorney General.

(*Continued from page 249*)

bination of n sampling units has a chance of being selected when sampling systematically. The emphasis here is on the word "combination," because in both schemes each sampling unit has the same weight and the same chance of selection. The difference in the two techniques can easily be seen in the following example.

Assume a population of 6 sampling units, numbered 1, 2, 3, 4, 5, and 6, and that it is desired to select 2 units for the sample. With simple random sampling, 2 random numbers less than or equal to 6 would be found in a table of random numbers to determine the units selected. One of the following 15 possible combinations would be chosen:

1, 2	2, 3	3, 5
1, 3	2, 4	3, 6
1, 4	2, 5	4, 5
1, 5	2, 6	4, 6
1, 6	3, 4	5, 6

With systematic sampling, a random start less than or equal to 3 would be found in a table of random numbers and every third sampling unit thereafter would be selected. One of the following 3 combinations would be chosen: 1 and 4, 2 and 5, or 3 and 6. Note that each sampling unit in a systematic sample can appear in only one combination of units. Each such combination may be considered a cluster. One cluster is selected in systematic sampling.

Simple random sampling

Simple random samples are generally not drawn for large-scale sample surveys. It is time-consuming and difficult to administer and control the selection and the use of n random numbers in a large-scale sample survey. However, the use of this technique, when it is beneficial from the cost standpoint, makes possible the calculation of an unbiased estimate of the contribution by a population or a stratum to the sampling variances.

Systematic sampling

Systematic sampling has two possible disadvantages. The first occurs when a population contains periodicities or is subject to trend. Systematic samples may then be ineffective in revealing the variation in the data. For example, assume that it was desired to sample enlisted personnel living on army posts in order to get data on motor-vehicle use. Further, suppose that a listing of names by barracks were available, that the name of a master sergeant generally headed the listing for each barracks, that 50 men were quartered in each barracks, and that the sampling rate were 1:25 with a random start of 1. The systematic sample would contain an excess of master sergeants whose characteristics, insofar as motor-vehicle use is concerned, would probably differ markedly from personnel of the lesser grades. This example also illustrates one of the criteria for cluster sampling—the need for making each cluster as heterogeneous as possible. As noted earlier, a systematic sample may be regarded as a single selected cluster. When a population contains periodicities and the sampling rate coincides to some extent with the period, the resulting sample is unduly homogeneous. The second disadvantage to systematic samples, usually surmountable, is that calculation of an unbiased estimate of the sampling variance is impossible with most designs employing this technique.

Systematic sampling has two important advantages. First, it is easy to administer; and second, if the arrangement of the population is such that the sampling units are essentially stratified, a systematic sample will be equivalent to a proportionate stratified sample with whatever gains in precision may be yielded by stratified sampling. For example, a file of reports may be arranged by counties within State highway districts. Sampling systematically from the file results in a sample that is stratified geographically.

Sampling with unequal probabilities

Underlying the techniques of simple random sampling and systematic sampling is the assumption that each sampling unit has approximately as much to contribute as any other sampling unit; hence, all units should be assigned equal weights. Also underlying these techniques is the second assumption that any one possible combination of sampling units is as acceptable as any other possible combination. When either of these two assumptions is grossly unrealistic, sampling with unequal probabilities is employed.

When the first assumption is not applicable, the technique most frequently adopted is to select sampling units with probability proportionate to some measure of the size of each sampling unit. Size does not necessarily mean volume or area. It means a function of the value of some characteristic, such as a total or the square root of a total, that is of interest in a sample study, or a function of the value of some other nonstudy characteristic that is highly correlated with the one being investigated. Sampling with probability proportionate to size often results in precision attained at a cost that is satisfactorily close to that yielded by an optimum design.

Sampling with probability proportionate to size has been practiced in the motor-vehicle-use studies conducted by some States. These studies are based on samples of dwelling units selected after several stages of sampling. In the first stage, all areas and places in a State are stratified. The estimated number of dwelling units in each area or place composing a stratum determines the chance for the selection of the area or place.

Problems of Nonresponse and Quality Control

In addition to the foregoing general treatment of some of the problems of sample design and sampling practices, two problems will now be discussed that apply as much to complete censuses as to sample surveys. These problems are nonresponse and quality control.

The larger the rate of nonresponse in any study, the more questionable the results of the study when applied to the entire population. It is often comfortably assumed that the nonrespondents have the same distribution of characteristics as respondents. Too often this assumption is not justified. Inferences of greater validity concerning the entire study population will be drawn if the nonresponse rate is reduced to such a level that, whatever the distribution of characteristics in the nonresponse group, it would not seriously have changed the distribution as estimated from the data received from the respondents. Research studies form the basis for decisions of some importance or they would not be conducted. The decisions should be based on relatively accurate data. A rule of thumb that has recently been applied by the Bureau of Public Roads in sample studies designed to obtain economic data is that the nonresponse rate should not be greater than 8 percent. To attain this rate or better can be costly, but possibly less costly than the results of decisions based on inaccurate estimates.

In modern sampling practice, considerable stress is given to the control of the quality of the final product. Sampling theory as described in this article and as presented in the literature aims at getting estimates of so-called true values with a given precision at minimum cost. Poor quality in questionnaire design, interviewer training, supervision, data processing, or any combination of these will generally yield different estimates of true value and precision than would have been obtained had a better quality of work been sought. Is a true value based on work of poor quality an acceptable outcome for a research study? The answer is obvious.

No inference is intended with respect to the quality of any past sample studies. The intent is to foster a greater awareness of procedural pitfalls that may beset researchers in areas other than sample design.

Variations in Stratified Sampling Procedure

Since stratified sampling has been adopted in many designs used by the State highway planning organizations, a closer look at two variations of this procedure may prove helpful. These variations are proportionate stratified sampling, in which the same proportion of sample cases is selected from each stratum, and optimum allocation, in which the number of sample cases to be selected from each stratum, not the proportion, is determined so as to yield optimum results. In both variations simple random sampling is used to select sampling units from each stratum.

Symbols will be used to generalize the exposition of the two variations. Some deductions will be drawn from the generalized expressions, and some explanation will be given. No attempt will be made to prove the formulas.[1]

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[1] For a more detailed explanation of these and of other aspects of sample design, the reader may refer to *Sample Survey Methods and Theory, Volume I, Methods and Applications*, by Morris H. Hansen, William N. Hurwitz, and William G. Madow, or to *Sampling Techniques*, by William G. Cochran. Both textbooks were published by John Wiley and Sons, Inc., New York, 1953.

:t:

N_h = Total number of sampling units in h^{th} stratum.

n_h = Number of sampling units selected from h^{th} stratum.

L = Total number of strata.

$N = \sum_h^L N_h$ = Total number of sampling units in the population.

$n = \sum_h^L n_h$ = Total sample size.

X_{hi} = Value of a characteristic of i^{th} unit in h^{th} stratum.

$X_h = \sum_i^{n_h} X_{hi}$ = Value of a characteristic for all units selected from h^{th} stratum.

$X = \sum_h^L \sum_i^{n_h} X_{hi}$ = Value of a characteristic for all sample units.

$\overline{X}_h = \frac{X_h}{n_h} = \frac{\sum_i^{n_h} X_{hi}}{n_h}$ = Mean of the characteristic for the sample units in h^{th} stratum.

$X'_h = N_h \overline{X}_h = \frac{N_h}{n_h} \sum_i^{n_h} X_{hi}$

= Estimate of a total for the h^{th} stratum.

$X' = \sum_h^L X'_h = \sum_h^L \frac{N_h}{n_h} \sum_i^{n_h} X_{hi}$ = Estimate of a total for the population.

$\frac{N_h}{n_h}$ = Reciprocal of the sampling fraction used in the h^{th} stratum.

$s_h^2 = \frac{\sum_i^{n_h} (X_{hi} - \overline{X}_h)^2}{n_h - 1}$

= An estimate from the sample of the square of the standard deviation, (the variance), of the population of sampling units in the h^{th} stratum.

$S_h^2 = \frac{\sum_i^{N_h} \left(X_{hi} - \sum_i^{N_h} X_{hi} \right)^2}{N_h - 1}$

= The variance of the population of sampling units in the h^{th} stratum.

y_{hi} = An estimate of the value of the i^{th} unit in the h^{th} stratum obtained from sources other than the sample.

$T_h^2 = \frac{N_h \sum_i^{N_h} (y_{hi})^2 - \left(\sum_i^{N_h} y_{hi} \right)^2}{N_h (N_h - 1)}$

= An estimate of the variance, S_h^2, of the population of sampling units in the h^{th} stratum obtained from sources other than the sample.

$s_{x'}^2 = \sum_h^L N_h^2 \frac{(N_h - n_h)}{N_h n_h} s_h^2$

= The square of the standard error of estimate of X'.

Since $n_1/N_2 = n_2/N_2 \ . \ . \ . = n/N$ in proportionate stratified sampling, a simplified formula for this design would be

$$s_{x'}^2 = \frac{N-n}{n} \sum_h^L N_h s_h^2$$

In proportionate stratified sampling, once n is known, the various n_h values are determined from $n_h = N_h\ (n/N)$. In optimum allocation, the various n_h values are determined by one of the two following formulas:

$$n_h = \frac{N_h S_h}{\Sigma N_h S_h} n \quad \text{or} \quad n_h = \frac{N_h S_h \div \sqrt{C_h}}{\Sigma (N_h S_h \div \sqrt{C_h})} n$$

Where:

C_h = The variable cost from stratum to stratum for collecting and processing the data.

The first allocation formula should be used in all cases except when the variation in the cost factors among the strata is of the magnitude of 3 or more, since it is the square root of the costs that enters into the formula.

Since proportionate stratified sampling uses a constant sampling fraction, it is relatively simple to produce estimates of population characteristics from the sample. This advantage is worth preserving unless optimum allocation produces a marked gain in sample efficiency. The following computations will aid in arriving at a choice between the two alternatives.

$$\sigma_{s_h}^2 = \frac{1}{N} \sum_h^L N_h (S_h - \overline{S})^2$$

or

$$\sigma_{T_h}^2 = \frac{1}{N} \sum_h^L N_h (T_h - \overline{T})^2$$

Where:

$$\overline{S} = \frac{\sum_h^L S_h}{L} \quad \text{and} \quad \overline{T} = \frac{\sum_h^L T_h}{L}$$

$$V_{S_h}^2 = \frac{\sigma_{S_h}^2}{(\overline{S})^2} \quad \text{or} \quad \frac{\sigma_{T_h}^2}{(\overline{T})^2}$$

When $V_{S_h}^2$ or $V_{T_h}^2$ equals one-third, optimum allocation will yield a squared standard error of estimate that is about 90 percent as large as that yielded by proportionate stratified sampling.

When $V_{S_h}^2$ or $V_{T_h}^2$ equals 1, optimum allocation will yield a squared standard error of estimate that is about 50 percent as large as that yielded by proportionate stratified sampling.

Therefore, it would be good practice to use proportionate stratified sampling when $V_{S_h}^2$ is of the magnitude of one-third or smaller; otherwise use optimum allocation.

Implicit in the preceding formulas is the assumption that good estimates are available for S_h, the population values of each stratum's standard deviation. When good estimates of the S_h values are not available, but it is possible to determine a measure of size, Y_h, for each stratum, and it is known that the Y_h values are fairly stable over the years, then a good approximation to optimum allocation can be determined by

$$n_h = n \frac{Y_h}{\sum_h^L Y_h}$$

So-called "optimum allocation" may yield less precise results than proportionate stratified sampling if good estimates of S_h values are not available or the Y_h values are not suitable.

Thus far, some formulas have been presented for determining the allocation of the total sample to each of the strata. No formulas have been given to calculate the total size of sample to be allocated. The following formulas may be applied in the case of proportional stratified sampling:

$$n_o = \frac{N}{d^2} \sum_h^L N_h^2 S_h^2 \text{ or } \frac{N}{d^2} \sum_h^L N_h^2 T_h^2$$

Where:

n_o = First estimate of total sample size.

d^2 = The square of the desired standard error of estimate.

If n_o/N is of the magnitude of one-tenth or greater, compute

$$n = \frac{n_o}{1 + \frac{n_o}{N}}$$

The following formulas may be used for optimum allocation:

$$n = \frac{\left(\sum_h^L N_h S_h \right)^2}{d^2 + \sum_h^L N_h S_h^2} \text{ or } \frac{\left(\sum_h^L N_h T_h \right)^2}{d^2 + \sum_h^L N_h T_h^2}$$

In area sampling especially, strata often vary considerably in size. A good rule when different sampling fractions are used for the several strata, as in optimum allocation, is to define strata measuring approximately equal in size. It may be found that some areas or clusters of sampling units are much larger than the generality. Each of these large areas may be set apart as a self-representing area composing a stratum. A satisfactory arbitrary rule to follow is to define a stratum with only one area or extra large cluster when the following relationship holds.

$$Y_{area} = \frac{Y}{2n}$$

Where:

Y_{area} = Measure of size for the area.
Y = Total measure of size.
n = Desired sample size.

Possible Application of Sampling Procedures

The theory and formulas presented in this article should find ready application in much of the data collection activities of highway planning organizations. One such activity may be singled out as an illustration, namely, the collection of data on local government finance. In most States, data are collected from the entire population of local governments. A stratified sample may be designed as an alternative.

Local governments may be stratified for example on the basis of total receipts, total expenditures, or the sum of the two. If some local governments levy one type of tax which others do not, this may serve as a basis for stratification. The relatively few large cities may be set up as self-representing areas. Values of T_h^2 for each stratum may be computed from data for the preceding year. The N_h values will be known. Either a proportional stratified sample or a sample using optimum allocation may be drawn, depending on the value of $V_{T_h}^2$. If it is decided to adopt optimum allocation, the allocation formula probably need not take the square root of cost, $\sqrt{C_h}$, into consideration.

The opportunity to improve the quality of the data should not be overlooked. It may be possible, with sampling, to modify forms and techniques to assure better quality.

Modification of procedure

One objection that has been raised to the application of sampling in this field is that some important items of local finance, such as bond issues, are too rare and unpredictable for a sample to result in acceptable estimates. This is true. However, it is also true that it is uneconomical to include the whole population in a study in order to obtain an acceptable value for a scarce item. The alternative is to find some inexpensive method for singling out such items for special treatment. One possible means is to inquire by mail or telephone to determine whether or not the rare event has occurred. Local authorities who reply in the affirmative may be asked to supply the data. The sample will yield information concerning all other matters of local finance.

Since the sample will be designed on the basis of the T_h^2 values, it would be well to collect data from the entire population periodically, possibly in 5- or 10-year intervals. Local governments grow or decline; some taxes gain or lose importance; new taxes are levied; and new demands on receipts are made. All of these may call for new stratification or new allocation, or both.

Application of method illustrated

The following hypothetical case will illustrate the numerical calculations involved in the use of the preceding formulas. Assume that a State has 104 incorporated places of all sizes, that 7 of these have 10,000 or more people each, and that the population of the next largest community is about 7,000. It is desired to estimate total highway receipts and expenditures, as well as various subclasses of receipts and disbursements, for all incorporated places. The decision is made to include the seven larger places and to sample the smaller places. It is also decided that the objective upon which to base sample size should be an estimate of total capital outlay plus maintenance costs for highways in which there is confidence that in 2 chances out of 3 the sample estimate for the remaining 97 places will differ no more than 5 percent from the true value. In the preceding year, when no sample was drawn, this value was $1,205 (000). Assuming that this amount has not changed radically, the desired standard error of estimate, d, would equal (0.05) (1,205) = 60.25. The value of d^2 would be 3,630.062. It is decided to stratify. Three strata are determined on the basis of the previous year's values of capital outlay plus maintenance expenditures. Table 4 gives the distribution of these values by strata.

Table 4.—Highway expenditures by incorporated places under 10,000 population in a hypothetical State

Highway expenditures for capital outlay and maintenance by incorporated places under 10,000 population		
Stratum 1, 63 incorporated places	Stratum 2, 24 incorporated places	Stratum 3, 10 incorporated places
$2,000	$10,500	$22,600
2,300	11,200	27,000
2,700	14,000	29,500
3,000	14,800	32,700
3,100	15,600	35,000
3,500	15,900	39,200
3,600	16,000	45,600
3,800	16,300	50,000
3,900	16,400	60,100
4,000	16,400	69,100
4,200	17,100	
4,300	17,200	
4,500	17,500	
4,600	17,700	
4,800	18,000	
4,800	18,100	
5,000	18,200	
5,000	18,300	
5,200	18,300	
5,200	18,500	
5,400	18,600	
5,500	18,600	
5,700	19,000	
5,800	19,500	
5,900		
5,900		
6,000		
6,000		
6,100		
6,200		
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9,600		
9,700		
9,700		
392,500	401,700	410,800
[1] (6,200)	[1] (16,700)	[1] (41,100)

[1] Average expenditures per incorporated place.

The formula and values for estimating the population variance of capital outlay plus maintenance expenditures for each stratum based upon the previous year's values, are:

$$T_1^2=\frac{\sum_{i}^{63}\left(y_{1i}-\frac{\Sigma y_{1i}}{63}\right)^2}{62}$$

$$=\frac{63\sum_{i}^{63}(y_{1i})^2-\left(\sum_{i}^{63}y_{1i}\right)^2}{(63)(62)}$$

. . . (Stratum

$$=\frac{63\,(2{,}683.81)-(392.5)^2}{3{,}906}$$

$$=\frac{15{,}023.78}{3{,}906}=3.8463$$

$T_1=1.96$ (estimate of the standard deviation).

$$T_2^2=\frac{24\sum_{i}^{24}(y_{2i})^2-\left(\sum_{i}^{24}y_{2i}\right)^2}{(23)(24)}$$

$$=\frac{24(6{,}842.35)-(401.7)^2}{552}$$ . . . (Stratum

$$=\frac{2{,}853.51}{552}=5.1694$$

$T_2=2.27$

$$T_3^2=\frac{10\sum_{i}^{10}(y_{3i})^2-\left(\sum_{i}^{10}y_{3i}\right)^2}{(9)(10)}$$

$$=\frac{10(18{,}907.12)-(410.8)^2}{90}$$

. . . (Stratum

$$=\frac{20{,}314.56}{90}=225.7173$$

$T_3=15.02$

The following calculations lead to a choice between optimum allocation and proportional allocation:

$\overline{T}=[1.96+2.27+15.02]\div 3=6.417$

$(\overline{T})^2=41.18$

$$\sigma_{T_h}^2=\frac{1}{97}\,[63(1.96-6.42)^2+24(2.27-6.42)^2+10(15.02-6.42)^2]$$

$$=\frac{1}{97}\,[2{,}406.1108]=24.8053$$

$$V_{T_h}^2=\frac{24.81}{41.18}=0.602$$

Therefore optimum allocation would be the preferred procedure.

A table similar to table 5 could be used to aid in the calculation of the sample size and in the allocation of the sample to the several strata.

General comments

The sample design was based upon one important characteristic of local finance, an estimate of total capital outlay plus maintenance costs for highways. Parenthetically, other characteristics that are highly correlated with the selected characteristic and which exceed it in dollar value, such as total highway expenditures, can be estimated from the sample with greater confidence than the selected design characteristic. Also, a different characteristic highly correlated with the design characteristic, if readily available, could have been used as the basis for stratification.

Seven incorporated places that could not conveniently be represented by other incorporated places were included with certainty in the total number of places to be selected. The remaining 97 incorporated places were grouped into three strata on the basis of the value of the design characteristic determined in the preceding year on a non-sample basis.

An estimate was calculated of the variance for each stratum on the basis of the previous year's value of the design characteristic. The square root of each of these values yielded an estimate of the standard deviation of the population of values in each stratum. The variance of these standard deviations about their average value was then calculated, and the ratio of this variance to the square of the average standard deviation was determined. Since this ratio was greater than one-third, optimum allocation was adopted rather than proportionate stratified sampling.

Table 5.—Data for estimating sample size and strata allocations

Stratum	N_h	T_h^2	$N_h T_h^2$	T_h	$N_h T_h$	$\frac{(N_h T_h)}{328.16}$	n_h
1	63	3.8463	242.3169	1.96	123.48	0.376	7
2	24	5.1694	124.0656	2.27	54.48	.166	3
3	10	225.7173	2257.1730	15.02	150.20	.458	8
Total	97		2623.5555		328.16	1.000	[1] 18

[1] $n=(328.16)^2/[3630.0625+2623.5555]=17.2$ (rounded to 18).

The calculations described in the preceding paragraph provided all values, except d^2, needed in the formulas for determining the total sample size and the allocation of this total among the three strata. The value of d^2 was fixed earlier when deciding on the sample design. It is the square of the range of values centered on the sample estimate which it is anticipated that the sample will yield. If the actual results do yield the design value of d, then the desired standard will have been attained.

Motor-Vehicle Size and Weight Limits

A comparison of State legal limits of motor-vehicle sizes and weights with standards recommended by the American Association of State Highway Officials is given in a table on pages 250–251. The statutory limits reported in this tabulation, prepared by the Bureau of Public Roads as of July 1, 1959, have been reviewed for accuracy by the appropriate State officials.

Statutory limits are shown for width, height, and length of vehicles; number of towed units; maximum axle loads for single and tandem axles; and maximum gross weights for single-unit trucks, truck-tractor semitrailer combinations, and other combinations.

Factors Influencing Mass-Transit and Automobile Travel in Urban Areas

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by WARREN T. ADAMS, Highway Transport Research Engineer

The objective of the research discussed in this article is to develop a relationship between the use of public and private transportation in urban areas and the principal factors influencing that use. If that can be satisfactorily done, an estimate can be made of the usage of each form of transportation under a given set of influencing factors and, as a consequence, estimates can be made of the total construction and operating costs involved in providing the required transportation system, including terminal facilities for each travel mode. The final step is to estimate the benefits of each plan or program in relation to cost.

By means of multiple regression analysis, data for 16 cities were used to develop a relationship between relative urban travel mode use and its principal influencing factors. Subsequently, this relationship was successfully tested through application of the derived equation to data received from five additional cities. The establishment of this relationship should materially aid in urban transportation planning, as the use of the equation makes it possible to predict relative transit use with an acceptable degree of accuracy.

A similar relationship between relative mode use and the principal influencing factors is also being investigated for major subdivisions of each urbanized area. Preliminary analysis indicates that the relationship for the subdivisions will be similar to that for the whole urbanized area.

WITHIN the next 25 years, there undoubtedly will be changes in modes of urban transportation. It is quite doubtful, however, that drastic changes will take place so rapidly that the basis for planning will be nullified overnight. At least so far as can be foreseen, the establishment of the relationship between the use of public and private transportation will permit the preparation of transportation plans on a more realistic basis.

A start has been made in establishing this needed relationship. The results of the investigation described here indicate that the approach will yield relationships, sufficiently accurate for transportation planning, that will enable planning officials to predict the travel mode split for a specific set of conditions.

In most urban areas since the late 1920's and excluding the depression and war years, the automobile has been supplanting public transportation as the mass carrier of people. The rate of change, however, has not been uniform from city to city or from year to year. In some metropolitan areas, principally the larger cities east of the Mississippi River, the decline in transit riding apparently is tending to level off. The trend from transit to automobile, however, has been continuing at either an accelerated or constant rate in many cities across the nation.

Public and private organizations have long been seeking a means for determining the change in both relative use and actual use of transit and automobile in urban areas. Transit and regulatory agency officials need this information to establish an equitable fare structure and to provide adequate transit service. Highway engineers have been faced with the necessity of determining whether the cost of street improvements, parking facilities, and transit facilities on highways will be justified by future use. Planning and municipal agencies are concerned with the interrelated effect of land-use distribution and population on the one hand, and the use of transit and automobile with the accompanying effect on public utilities on the other. Municipal authorities must also determine the impact on the tax structure of relative travel mode use, especially if transit operations should become the responsibility of a public agency. Primarily, all of these groups are concerned with the factors and elements that cause variations in mode of travel and their relation to the economic and political system.

Study Procedure

In previous studies, attempts were made to establish relationships between transit-riding habits and several related factors. In general, these factors were population, automobile registrations, transit service, economic welfare, and transportation costs. The studies, however, have not yielded conclusive results. They did not develop relationships that would make it possible to forecast relative mode use with an acceptable degree of accuracy.

In the last few years, more attention has been given to land use as one of the principal factors affecting urban transportation. Studies have indicated an appreciable degree of correlation between travel mode on the one hand, and residential, commercial, and industrial land use on the other.

In this study, a land-use distribution factor has been combined with factors relating to population, automobile ownership, employment, dwelling units, transit-service ratio, and size of urbanized area in order to develop a basis for forecasting travel mode use.

Source of Data

The home-interview origin-and-destination survey reports for all cities completed since January 1948 were reviewed; cities not having transit service were naturally eliminated. Cities in which surveys were made prior to 1948 were not considered, principally because war-caused distortions still exerted a major influence through 1947, and the quality of many of the early surveys was questionable. Of the 30 cities studied, 8 surveys were completed in 1948, 7 in 1949, 6 in 1950, 3 in 1951–52, and 6 in 1953.

Through the cooperation of the American Transit Association, information on transit service at the time of the origin and destination survey was obtained from the transit companies in 22 of the 30 cities. Since previous studies had indicated that one of the key factors in any developed relationship might be land-use distribution, such information corresponding to the origin-and-destination survey period was also requested from the planning agencies of each city.

Methods of Analysis

Simple and multiple regression equations were developed and tested in the study of the relationship of travel mode to various factors. Among the simple factors examined were population, automobile ownership, trips to work, and total survey land area. Among the compound factors investigated were population density, automobile ownership per capita, and employment per capita. Also tested were the regression relationships between travel mode split and factor combinations, including various compound factors such as employment and automobile ownership per capita. None of these relationships yielded either an acceptable standard error of estimate or a high degree of correlation.

Symbols and definitions

Letter symbols used to represent the various factors discussed in subsequent sections of this article are defined as follows:

$_1A_c$ = Commercial and industrial land area (sq. mi.) within a 1-mile radius of the CBD center.

A_{tc} = Commercial and industrial land area (sq. mi.) within the entire urbanized area.

a = Number of automobiles owned.

D = Parking demand as related to the number of types of employment.

E = Economic factor as determined by $(P/e)^{3.5}(h/e)^{1.5}(P/a)^{1.0}(h/a)^{1.5}$.

e = Number of employees going to work on an average weekday.

F = Terminal or parking facility factor.

h = Number of dwelling units.

M = Urbanized land area in square miles.

P = Population over 5 years of age in the survey urbanized area.

R_{1c} = Mean distance of commercial and industrial land within a 1-mile radius of the CBD center.

R_{tc} = Mean distance from the CBD center of commercial and industrial land within the entire urbanized area.

R_p = Mean distance of center of population from the CBD center.

R_u = Mean distance of entire urbanized area from the CBD center.

r_1 = Land-use distribution ratio, $1-(R_p/R_u)$.

r_2 = Land-use distribution ratio, R_pR_{tc}/R_u.

r_3 = Land-use distribution ratio, A_{1c}/A_{tc}.

r_4 = Land-use distribution ratio, $(A_{tc}/A_{tc}R_{tc}) = (1/R_{tc})$.

r_5 = Land-use distribution ratio, $(A_{1c}R_{1c}/A_{tc}R_{tc})$.

S_r = Ratio of the square root of the average vehicle speed for the different travel modes.

T = Transit-service ratio factor as determined by $[V^{1.0}/P^{1.50}M^{0.25}](S_r)(F/D)$.

U = Land-use distribution factor as determined by $(r_1)(r_4)(r_5)$.

V = Equivalent revenue vehicle-miles operated per weekday during the survey.

y = Reported percentage of total person trips made via transit.

y_1 = Estimated percentage of total person trips made via transit.

Development of equations

As transit service information was received, tests were made to determine whether there might be a significant relationship between transit service data and travel mode split. As the lone independent variable, transit service did not produce a satisfactory estimating equation. Using a transit-service ratio factor in conjunction with a population factor, a combination automobile and employment factor, and an urbanized land-use factor, semilog multiple variable equations of the form $y_1=A+b_1 \log P+b_2 \log E+b_3 \log T+b_4 \log M$ were developed that gave the results shown in figure 1 for the cities with available data for 1948, 1949, and 1950.

These results indicated that there might be at least one other factor which, if included, would produce the relationship sought regardless of the year. From previous studies, it was believed that this factor might be largely based on land-use distribution, for with each succeeding year since 1948 there has been an increasing decentralization of residential, commercial, and industrial land use with respect to the central business district (CBD). Tests were made of ranges of various ratios and combinations of ratios involving distribution of land used commercially and industrially within and about the CBD, population distribution with respect to the center of the urbanized land area, and population distribution with respect to employment location. Although there were differences in the amount of variation explained, nearly all combinations tended to reduce the year-to-year variations. Using the semilog equation $y_1=A+b_1 \log P+b_2 \log E+b_3 \log T+b_4 \log U+b_5 \log M$, the land-use distribution factor, U, brought the 16 cities listed in table 1 into a straight-line relationship as shown in figure 2.

As additional information is obtained, the land-use distribution factor as well as other factors in use may require modifications.

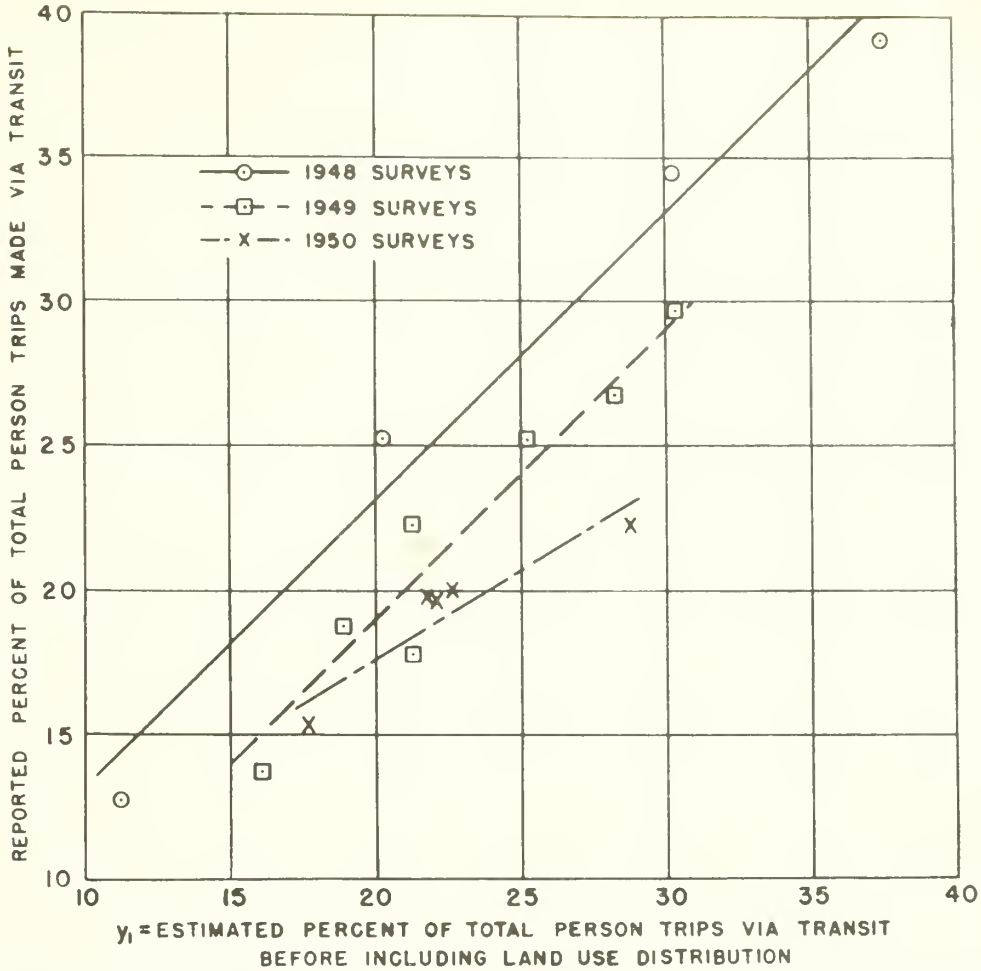


Figure 1.—Relationship of reported relative transit use to that derived from estimating equation, before inclusion of the land-use distribution factor.

Table 1.—Basic data for 16 cities used in developing relative transit use equation for entire urbanized area

O-D survey year and urbanized area	Population over 5 years of age P	Dwelling units h	Employees going to work per average weekday e	Automobiles owned a	Equivalent revenue vehicle-miles per average weekday V	Urbanized land area M	Land-use distribution factor U	Reported relative transit use y
	Thousands	*Thousands*	*Thousands*	*Thousands*	*Miles*	*Sq. mi.*	*Ratio*	*Percent*
1948:								
Washington, D.C.	992.6	336.2	380.8	203.5	155,060	108.80	0.00456	39.3
Allentown-Bethlehem, Pa.	156.4	48.8	61.6	28.6	12,490	18.95	.00628	32.1
Tacoma, Wash.	125.0	48.0	36.1	35.2	11,960	45.80	.03350	25.2
Tucson, Ariz.	113.7	38.7	29.6	32.9	3,610	41.00	.07398	12.7
1949:								
Albuquerque, N. Mex.	100.8	34.9	30.4	27.5	4,510	38.93	.02035	13.6
Madison, Wis.	94.3	33.4	29.6	25.3	5,780	20.40	.03034	22.4
Racine, Wis.	69.5	23.3	23.1	18.5	3,210	11.56	.06204	17.8
Sharon-Farrell, Pa.	44.3	13.7	12.9	9.4	1,530	8.08	.19520	18.5
1950:								
Dallas, Tex.	471.1	168.1	182.5	153.8	57,900	173.20	.00253	19.8
Rockford, Ill.	102.5	36.2	48.7	33.1	6,020	24.54	.05248	15.1
Altoona, Pa.	77.5	24.1	17.1	16.8	3,680	15.82	.03204	20.0
Kenosha, Wis.	50.2	17.3	21.7	13.4	2,580	8.32	.19264	19.6
1953:								
Detroit, Mich.	2,642.2	875.4	971.1	845.8	183,170	548.85	.00004	16.7
Houston, Tex.	765.9	272.7	278.8	256.3	48,020	244.50	.00024	13.6
San Diego, Calif.	513.0	175.0	152.1	250.5	36,170	152.01	.00025	13.2
Stockton, Calif.	124.3	41.6	34.9	70.0	3,000	31.89	.01109	7.8

Variations in some factors have been investigated, and several satisfactory estimating equations with only minor variations have been developed. All equations have yielded a standard error of estimate of less than 1.5 percentage points, and several have yielded less than 1.0 percentage point of the reported transit use as a percentage of total person trips for the entire urbanized areas of the 16 cities. Transit usage as reported in table 1 for these cities ranged from 8 to 40 percent of the total trips made by persons, with a mean of approximately 20 percent. Thus, a standard error of 1.5 percentage points is equivalent to 7.5 percent of the mean value of 20 percent for total transit trips per weekday.

Of the semilog, multiple regression equations developed, the following equation was used in this study: $y_1 = -2.6466 + 3.7084 \log P + 0.3912 \log E + 2.3757 \log T + 0.4918 \log U - 0.9708 \log M$. The basic data for this equation are found in table 1.

Since developing the estimating equation, complete information has been obtained from 5 cities in addition to the 16 cities listed in table 1. Applying the equation to the data for the five cities, the results, as illustrated in figure 3, were found to be within the previously stated standard error of estimate.

Discussion of Factors

In the estimating equation, the three compound factors—economic, transit service, and land use—have been developed through testing variations of each over a range that would determine the maximum effect of each variable in correlation with other potential variables. As more information is obtained from the cities mentioned in this article as well as additional cities, and as this information is adapted to electronic computer programing, it is anticipated that more precise parameters will be established.

Economic factor, *E*

Apparently there is a high degree of correlation between relative use of each transportation mode and some economic factor. Many contend that this factor is either income or wealth. But what income or wealth? Is it gross or net? What should be included and what deducted? Moreover, how could accurate measurements of income or wealth be made? Correlating travel mode use and related origin and destination information with sufficiently accurate income or wealth data will be most difficult under legal restrictions applying to the release of the latter.

There may be other economic items that have a higher degree of correlation with travel mode than the ones used in this study—population, number of dwelling units, number of automobiles owned, and number of employees going to work on an average weekday. These four items seem to be the best available that can be accurately measured. The simple linear correlation coefficient relating the various compounds of the components of *E* (economic factor) with relative mode use varied between 0.40 and 0.60.

The use of both population and number of dwelling units in relation to the number of automobiles owned and the number of employees going to work per average weekday may be challenged. The correlation obtained by use of these factors in combination has been greater than when only one factor has been used. This may be due to compensating errors in the origin and destination surveys, and to the effect of differences in population per dwelling unit.

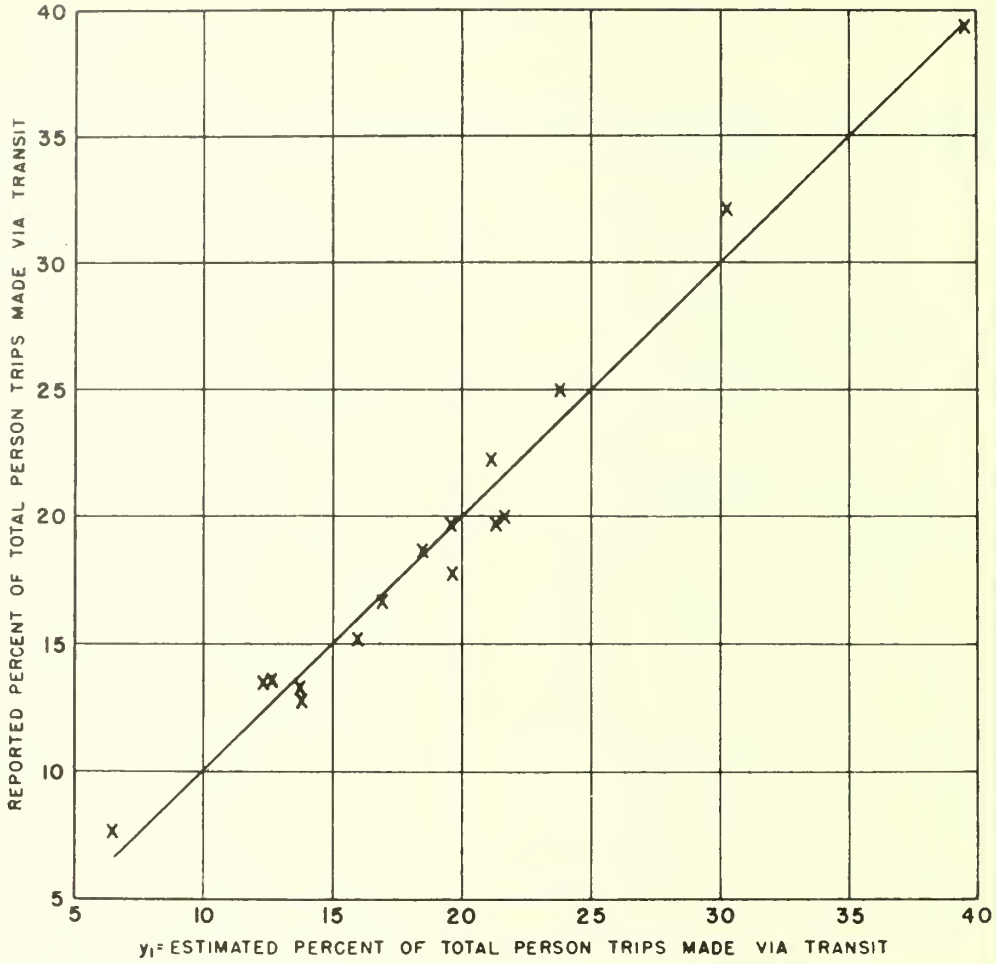


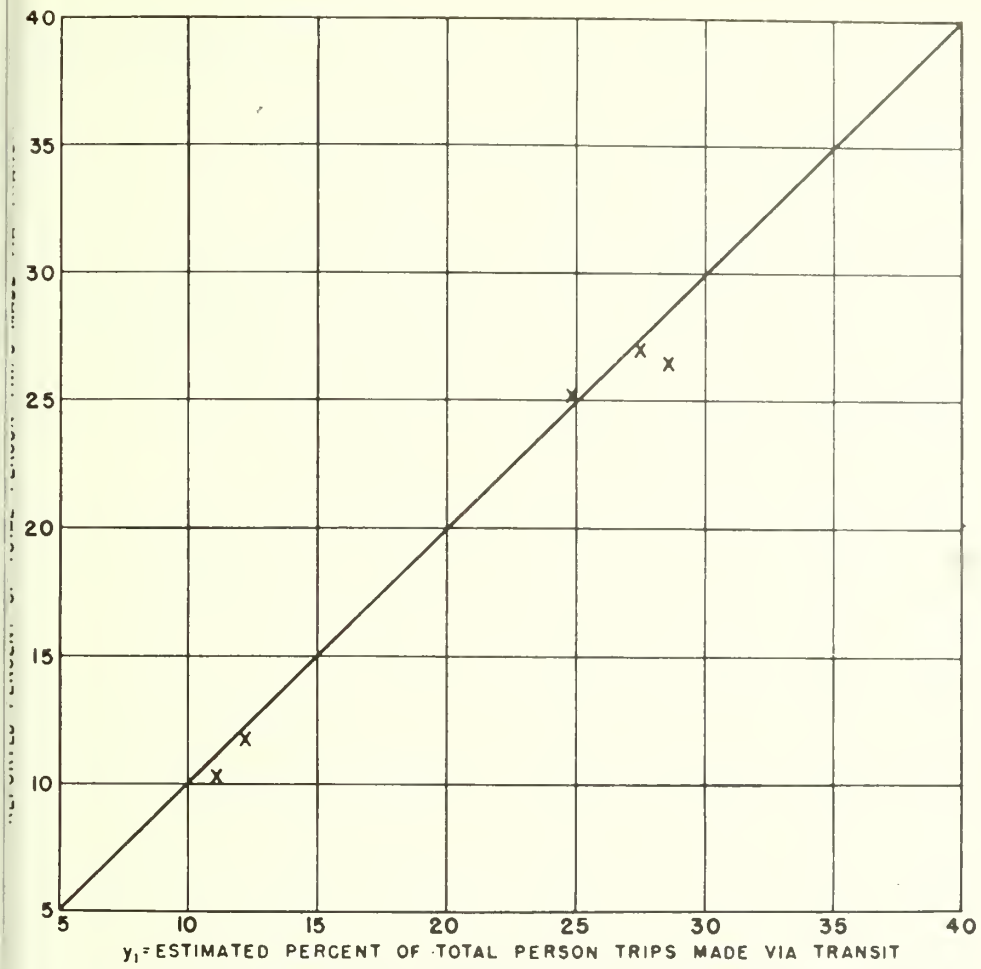
Figure 2.—Relationship of reported relative transit use to that derived from the estimating equation, after inclusion of the land-use distribution factor.

Transit-service ratio factor, *T*

Although not approaching unity, there is a significant degree of correlation between the developed transit service variable and the dependent transit use variable. The simple linear correlation coefficient for the transit-service ratio factor and relative transit use has ranged from 0.30 to 0.45. The degree of correlation not only varies from city to city, but also within many cities there is likely to be an even greater variation among subdivisions (sectors, districts, etc.) or transportation channels. Furthermore, the effect of each of the components in the transit service factor apparently varies among cities and their subdivisions.

Equivalent revenue vehicle-miles, V.—Equivalent revenue vehicle-miles operated per weekday are expressed in terms of a 50-seat bus revenue-mile. Included are all revenue vehicle-miles operated per weekday, regardless of the number of passengers carried on each vehicle trip. This item has been derived by applying a carrying capacity factor to the average weekday revenue vehicle-miles recorded during the survey. The capacity factor has been developed through assignment of each vehicle size, by time periods, in proportion to the ages of the active vehicle groups. Inasmuch as it is impossible to obtain actual average carrying capacity during the surveys without a prior uniform arrangement with the transit operators, this derivation gives an arbitrary, but uniform, estimate for all cities that most nearly approaches the "true" average.

Average vehicle-speed ratio, S_r.—The ratio of the square root of the speeds of automobile and transit vehicle travel is one of the components of the transit-service ratio factor. There are few who do not consider this ratio an influencing item in determining the travel mode split. Yet based on available data, the standard error is increased by only 0.1 percent when the speed ratio factor is excluded. This would indicate that the variation of this ratio from city to city is not appreciable with respect to the entire urbanized area for each city. Due to the limited amount of data available and the apparent relatively small effect on the standard error, the speed ratio factor has not been included in the estimating equation adopted for this article. The investigation so far has borne out that the use of the ratio of the square root of the speed of each travel mode, although



gure 3.—Relationship of reported relative transit use to estimated relative use in five cities.

ducing the sensitivity of the component, ap- ars to be most satisfactory.

The limited work done to date by subdivi- n of urban areas has pointed to a much eater influence of the speed component for bdivisions and transportation channels than r entire urban areas. This is accounted for the greater spread of relative speeds within e subdivisions. The results still indicate, wever, that the ratio should be based on the uare root of the respective speeds.

It is quite possible that additional data may tablish that different ratios should be ap- ied to the two principal components of over- travel time for each travel mode, namely, hicle speed and the terminal factor. The ect of these components must be more ac- rately determined not only to estimate ansit and automobile usage under specific nditions, but also to develop the required ansit, parking, and highway capacities with tendant capital and operating costs for the timated use of each travel mode.

nd-use distribution factor, *U*

The land-use distribution factor is a complex e that has been developed from a series of udies within the limitations of available ma- rial, time, and computing equipment. In lating this factor to travel mode use, the nple linear correlation coefficient varied m 0.60 to 0.75. It appears likely that ore efficient analysis of present and future data by means of an electronic computer program will produce either more precise values for the factors now being used or simpler factors that may prove to be more satisfactory. For the entire urban area and for the subdivisions investigated, there appear to be five land-use distribution ratios about the CBD center that should be considered. These ratios are listed on page 257 as r_1, r_2, r_3, r_4, and r_5.

In arriving at the mean distance to the CBD center from the centroid of each of the items included in the land-use ratios, the same procedure was used for each item. Therefore, a detailed description of the derivation of one item (commercial and industrial land use) will suffice for all. Each city was divided into four quadrants by rectangular coordinate axes passing through the CBD center. In each quadrant, the area of each industrial and commercial parcel, or each group of adjacent parcels, actually used for one of these purposes at the time of the origin and destination survey was multiplied by the distance from the CBD center to the centroid of the parcel or group of parcels. These products were then summed for the four quadrants, and this summation was divided by the summation of the areas of all the industrial and commercial parcels in the urbanized area.

In some instances it was found to be more efficient to determine the mean distance through summing the products obtained by multiplying the areas by their distances to the two coordinate axes, and then extracting the square root of the sum of the squares of those two product summations.

In the estimating equation adopted, ratios r_2 and r_3 were not included in the land-use distribution factor. Studies which have not been concluded indicate, however, that the inclusion of ratios r_2 and r_3 with possible modification of the other ratios would reduce the standard error of estimate.

The present study has shown that it is not necessary to differentiate between commercial and industrial land for the entire urban area. This is apparently due to the balancing effect of the two land uses over a complete urban area. Within highly specialized subdivisions or transportation channels serving predominantly one type of land use, the investigation shows that the two uses will have to be considered separately. It may even be necessary to subdivide the two classifications into four classes: industrial, office, shopping durable, and shopping service and convenience. Based on probable accuracy of land-use forecasts, it is questionable whether further breakdowns can be justified for transportation channel subdivisions and even much less justifiable for subdivisions comparable to origin and destination districts. To justify more classifications of land use, a much greater specialization of land use in a transportation channel subdivision would be required than has been found or seems probable in the future.

Urbanized land area factor, *M*

Defining urbanized area boundaries is of utmost importance in determining the relative use of transit vehicles and automobiles. Urbanized areas, as used in the present study, have been confined to contiguously developed land, and future estimates must be based on anticipated contiguously developed land. Furthermore, such land must have a minimum residential population per area unit of 500 persons per square mile, or a minimum number of 2,000 trip-ends per square mile to be included as part of the urbanized area. Islands of vacant land should be included if the land outside is sufficiently developed to bring the combination of vacant land and adjacent, outside developed land up to the minimum requirements just stated. Pockets of vacant land at the origin and destination boundary not meeting these specifications should be excluded.

Many subdivisions with population or trip-ends above the minimum requirements cannot be served by transit unless the service costs are partially defrayed by either the subdivision, the entire urban area, or the entire transit system. In border subareas where either the residential population or number of trip-ends is less than the minimum, the only mode of urban transportation will be the privately owned automobile unless transit service is furnished by an intercity carrier, or it is almost entirely underwritten on a service charge basis due to relatively low transit use. If border land with subminimum population or trip-ends is included in the study, the urban-

ized area factor, M, would be appreciably changed.

Work in Progress

Of the factors investigated so far, the three that contribute most in explaining the variance are the transit-service ratio factor, the land-use distribution factor, and the economic factor. Some of the factors are still being studied to determine whether they should be modified or replaced. It is possible that the estimating equation may be appreciably changed as a result of these continuing studies. If the work in the past, however, is a criterion for the future, the estimating equation will not likely be altered significantly.

Expansion of project

In addition to continuing research on the whole metropolitan area, this study is now being expanded in two directions. Generalizing, it can be stated that the present estimating equation is an expression of the division of trips between transit and automobile in relation to factors pertaining to the home area (employment, automobile ownership, and population distribution), and to factors applying to the entire metropolitan area (land-use distribution, transit-service ratio, total population, and urbanized land area). The equation is presently being tested to determine whether it, or a modification thereof, will apply to subdivisions of each metropolitan area. At the present time, sufficient information to investigate such application has been obtained from only three cities.

The results so far indicate that the equation, after modifying the land-use and transit-service ratio factors, will make it possible to forecast with acceptable accuracy the split between transit and automobile trips for each subdivision. Two items that apparently have more influence on the travel mode within each subdivision than for the whole urban area are the average ratio of overall trip time by the two transportation modes, and the ratio of commercial to industrial land use. Since only 20 subdivisions have been partially investigated, a precise basis for modifying the transit-service ratio and land-use factors has not yet been developed.

The other extension of the project has been an attempt to determine the influence on the estimating equation of other factors in destination areas. In this subphase, even less information is available. A limited amount of information on CBD destination factors has been gathered in several cities, and the relation between these factors and the travel mode split has been tested. Foremost among the items that apparently should be introduced into the equation is a parking facility factor. The equation, modified by this factor, appears to yield a low standard error of estimate in predicting the travel mode split in destination areas. The factor, however, is not confined to total parking space in each destination area; it also includes accessibility to demand as expressed by a relationship including parking charges and walking and parking time.

Areas of future research

Only the surface has been scratched in attempting to establish factors and estimating equations pertaining to the travel mode split by home and destination zones. Relationships by subdivision should also be developed for the peak travel period. Some work is under way on this peak period relationship for the entire metropolitan area.

The results of research accomplished so far seem to indicate that the speed factor, in general, varies as the ratio of the square root of trip speed. Convenience and irritation items, to the extent that it has been possible to measure them, and modified somewhat by cost, are apparently as important, if not more so, than absolute vehicle speed. This observation, however, may not hold for freeway and rapid-transit operation. In fact, testing additional data for vehicle operation on unrestricted rights-of-way may alter the findings in this field. The analysis of the Chicago origin and destination survey should yield much information on this phase. It is the only city with both rapid transit and a limited amount of freeway traffic data now available for testing.

To carry out further analyses of relationships, much additional information will be needed. Many cities and transit companies have cooperated. If the needed relationships are to be established, it will be necessary to call on these and other cities for more basic data from time to time. Much of the information should be gathered at the time of the origin and destination survey; in fact, it should be made a part of it. It will take time and money to gather and assemble the information, but it should be beneficial to both city planning agencies and transit companies.

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VOL. 30, NO. 12

FEBRUARY 1960

Public Roads

A JOURNAL OF HIGHWAY RESEARCH

PUBLISHED
BIMONTHLY BY THE
BUREAU OF
PUBLIC ROADS,
U.S. DEPARTMENT
OF COMMERCE,
WASHINGTON

A recently completed Federal-aid primary facility through Rio Piedras, south of San Juan, Puerto Rico.

IN THIS ISSUE

Forecasts of Population, Motor-Vehicle Registrations, Travel, and Fuel Consumption . . . 261

Estimated Travel by Motor-Vehicles in the United States, 1958 275

Common-Carrier Passenger and Freight Services Available to Communities on the Interstate Highway System 276

Surface and Subsurface Temperature Variations and Comparisons 283

U.S. DEPARTMENT OF COMMERCE
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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

Published Bimonthly

Vol. 30, No. 12 *February 1960*

E. A. Stromberg, *Acting Editor*
C. L. Fine, *Assistant Editor*

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Washington 25, D.C.

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Use of funds for printing this publication has been approved by the Director of the Bureau of the Budget, March 28, 1958.

Contents of this publication may be reprinted. Mention of source is requested.

Forecasts of Population, Motor-Vehicle Registrations, Travel, and Fuel Consumption

BY THE OFFICE OF RESEARCH
BUREAU OF PUBLIC ROADS

Reported by THOMAS R. TODD, Head,
Revenue Planning Section,
Highway Needs and Economy Division

According to forecasts made by the several State highway departments (excluding Alaska and Hawaii), there will be 230 million people living in the United States in 1976. During that year, these people are expected to register 114 million motor vehicles, which will travel 1.2 billion vehicle-miles. The estimates thus predict a 37-percent increase in population over 1956, the base year of the forecasts; increases of 75 percent in motor-vehicle registrations and 93 percent in travel are expected. The forecasts, prepared by the States for the Bureau of Public Roads Highway Cost Allocation Study, are based, in general, upon the continuation of recent trends in population, living standards, density of motor-vehicle ownership, and characteristics of motor-vehicle use. Significant changes in any of these basic trends during the period involved could be expected to have marked effects upon the accuracy of these forecasts.

THE Highway Cost Allocation Study, undertaken by the Bureau of Public Roads at the request of Congress pursuant to section 210 of the Highway Revenue Act of 1956, has as its objectives an assessment of highway needs and the collection and analysis of other information. On the basis of these assessments, Congress may determine what taxes for highway improvement should be imposed by the Federal Government and how they may be equitably distributed among beneficiaries of Federal-aid highways. State and nationwide forecasts of motor-vehicle registrations and travel and of motor-fuel consumption were needed as a basis for forecasting highway needs and revenues. In order that such predictions might be prepared from a sound background, it was also necessary that population forecasts be developed. Such forecasts of population, motor vehicles and their use, and fuel consumption, as prepared by the States, are presented in this article.

The national summaries, compiled from the estimates made by the States, constitute what is believed to be a reasonably accurate prediction of what the future use of the Nation's highways will be. Forecasts by some of the States might be considered as being too conservative; forecasts of others as too optimistic. Perhaps a more critical appraisal could be taken of the conservative forecasts than of the optimistic ones. When considered on a national scale, however, divergencies are probably largely cancelled out.

Since Alaska and Hawaii had not achieved statehood when these studies were originated, predictions for those jurisdictions were not included in the summary trend forecasts presented here. However, Hawaii, as well as Puerto Rico, provided projections, which are included in all tables depicting individual State forecasts.

Forecast Methods

In conformity with the needs of the Highway Cost Allocation Study, and preparatory to estimating highway needs, the State highway departments were requested to prepare year-by-year forecasts of highway travel through 1976 with extrapolations to 1981, 1986, and 1991.[1] Suggestions and guides concerning the preparation of forecasts were distributed to the States by the Bureau of Public Roads. Various aids to forecasting, such as the Bureau of the Census estimates of future population by States, were also provided the highway departments. However, the only requirements imposed upon the States were that the forecasts submitted be reasonable in the light of past and current trends, and that State highway officials be prepared to stand behind them.

The method used to prepare the forecasts were, in general, the same as had been used in preparing those requested for the Nationwide Highway Finance Study of 1954.[2] Consequently, the projections prepared in 1957 were similar to those prepared for the earlier study, though generally somewhat higher—and, it should be noted, the newer forecasts

Table 1.—Population, motor-vehicle registrations, travel, and motor-fuel consumption in the United States (excludes Alaska and Hawaii), 1921–56 and State forecasts for selected years, 1961–91 [1]

Year	Population [2]	Motor vehicles registered [3]	Vehicle-miles traveled	Gallons of motor fuel consumed
	Thousands	*Thousands*	*Millions*	*Millions*
1921	104,541	10,494	56,681	3,935
1922	110,055	12,274	68,340	4,841
1923	111,950	15,102	84,045	6,078
1924	114,113	17,613	102,423	7,497
1925	115,832	20,069	119,057	8,749
1926	117,399	22,200	135,905	10,064
1927	119,038	23,303	150,533	11,331
1928	120,501	24,689	167,317	12,361
1929	121,770	26,705	188,617	14,139
1930	123,077	26,750	199,263	14,754
1931	124,040	26,094	203,777	15,457
1932	124,840	24,391	190,728	14,339
1933	125,579	24,159	188,784	14,348
1934	126,374	25,262	204,070	15,415
1935	127,250	26,546	215,428	16,345
1936	128,053	28,507	235,205	18,099
1937	128,825	30,059	253,818	19,455
1938	129,825	29,814	257,087	19,612
1939	130,880	31,010	271,379	20,714
1940	131,954	32,453	288,155	22,001
1941	133,417	34,894	312,307	24,192
1942	134,670	33,004	259,990	19,940
1943	134,697	30,888	207,887	16,004
1944	134,075	30,479	213,066	16,430
1945	133,387	31,035	245,145	19,149
1946	140,678	34,373	328,431	25,649
1947	144,261	37,704	360,689	28,244
1948	146,421	40,960	387,209	30,447
1949	148,578	44,448	413,597	32,456
1950	150,910	48,945	451,771	35,604
1951	153,440	51,643	484,582	38,207
1952	155,957	52,966	512,689	40,592
1953	158,572	55,939	539,199	42,809
1954	161,087	58,219	558,801	44,322
1955	164,360	62,343	595,856	47,780
1956	167,250	65,119	622,932	50,011
1961	180,656	77,002	753,764	60,690
1966	195,353	89,161	898,691	72,605
1971	211,653	101,240	1,051,412	85,073
1976	229,758	113,642	1,200,263	97,141
1991	---	-------	1,733,602	----

[1] Data are summaries of estimates prepared by the States for population, registrations, and fuel consumption for 1947–76 and for travel for 1921–91.
[2] Excludes armed forces overseas.
[3] Includes publicly owned vehicles.

[1] Extensive data for 1957, and summaries of the forecasts, have been published in the *Third Progress Report of the Highway Cost Allocation Study*, House Doc. No. 91, 86th Cong., 1st sess.

[2] *Needs of the Highway Systems, 1955–84*, House Doc. No. 120, 84th Cong., 1st sess., 1955.

were made with much greater care. It was believed that the higher levels forecasted were justified by the fact that actual figures for motor-vehicle registrations, travel, and fuel consumption that had become available for 3 years (1955–57) since the earlier forecasts were prepared had been, in almost every instance, slightly higher than the corresponding values previously forecasted. Other factors believed to justify more optimistic forecasts were higher population estimates made by the Bureau of the Census and the inclusion in the new forecasts of estimates for publicly owned vehicles.

Summarization of the individual State forecasts made for the Highway Cost Allocation Study produced reasonable nationwide estimates, as table 1 and figure 1 indicate. Each of these forecasts of population, vehicle registrations, motor-vehicle travel, and motor-fuel consumption, when coupled with historical data for the same comparable series for 1921 through 1956, indicates a trend commensurate with that exhibited in recent years.

Population Forecasts

Forecasts of population made by the States indicate that the 1976 population of the United States, excluding Alaska and Hawaii, will be about 230 million inhabitants. This forecast, considered rather optimistic in 1957, has been made to appear reasonably conservative by later projections prepared by the Bureau of the Census.[3] The four Census projections range from a high of 244 million to a low of 216 million for 1975. This places the forecast used in this report at about the midpoint of the Census Bureau projections. It is also anticipated that by 1976 there will be 154 million persons in the driver age group, 15–74. As this estimate is somewhat less, on a percentage basis, in respect to total population than the 1956 estimate, it also must be considered as being reasonably conservative. Therefore, forecasts by the States of both total population and potential drivers for 1976 appear to qualify as reasonably reliable foundations on which to base estimates of motor-vehicle ownership ratios and registrations.

Trends by census divisions

Table 2 summarizes the State population forecasts by census divisions for 1956 and 1976. Figure 2 portrays graphically the population estimates from 1947 to 1976. Detailed forecasts by States are included in table 3.

The Pacific division shows the largest expected increase during the 20-year period, both numerically (16.5 million) and relatively (95 percent). The Mountain division is expected to have the second highest percentage increase (65), although the numerical increase (4.0 million) is relatively small. The South Atlantic (South) division is the only other geographic area in which the anticipated percentage increase (49) is above the national average of 37 percent. The East South Central division is expected to have a population increase of only 12 percent (1.4 million persons) during the forecast period. The West North Central division is next in order with an anticipated 17-percent increase.

Although, large numerical increases in population are expected in each of the three major regions of the Nation, only the Western region is expected to gain in relative position, from 14.1 percent of the total population in 1956 to 19.2 percent in 1976, an increase of 36 percent. The Northern region is expected to change from 54.9 percent of the total population in 1956 to 50.9 percent in 1976, a decline of 7 percent. A more moderate change in the Southern region, from 31.0 to 29.9 percent of the total, represents a decline of 4 percent. The Northern region includes the New England, Middle Atlantic, East North Central, and West North Central census divisions; the Southern region consists of the South Atlantic (North), South Atlantic (South), East South Central, and West South Central census divisions; and the Mountain and Pacific divisions make up the Western region.

[3] *Current Population Reports*, Bureau of the Census, Population Estimates, Series P-25, No. 187, November 1958, p. 2.

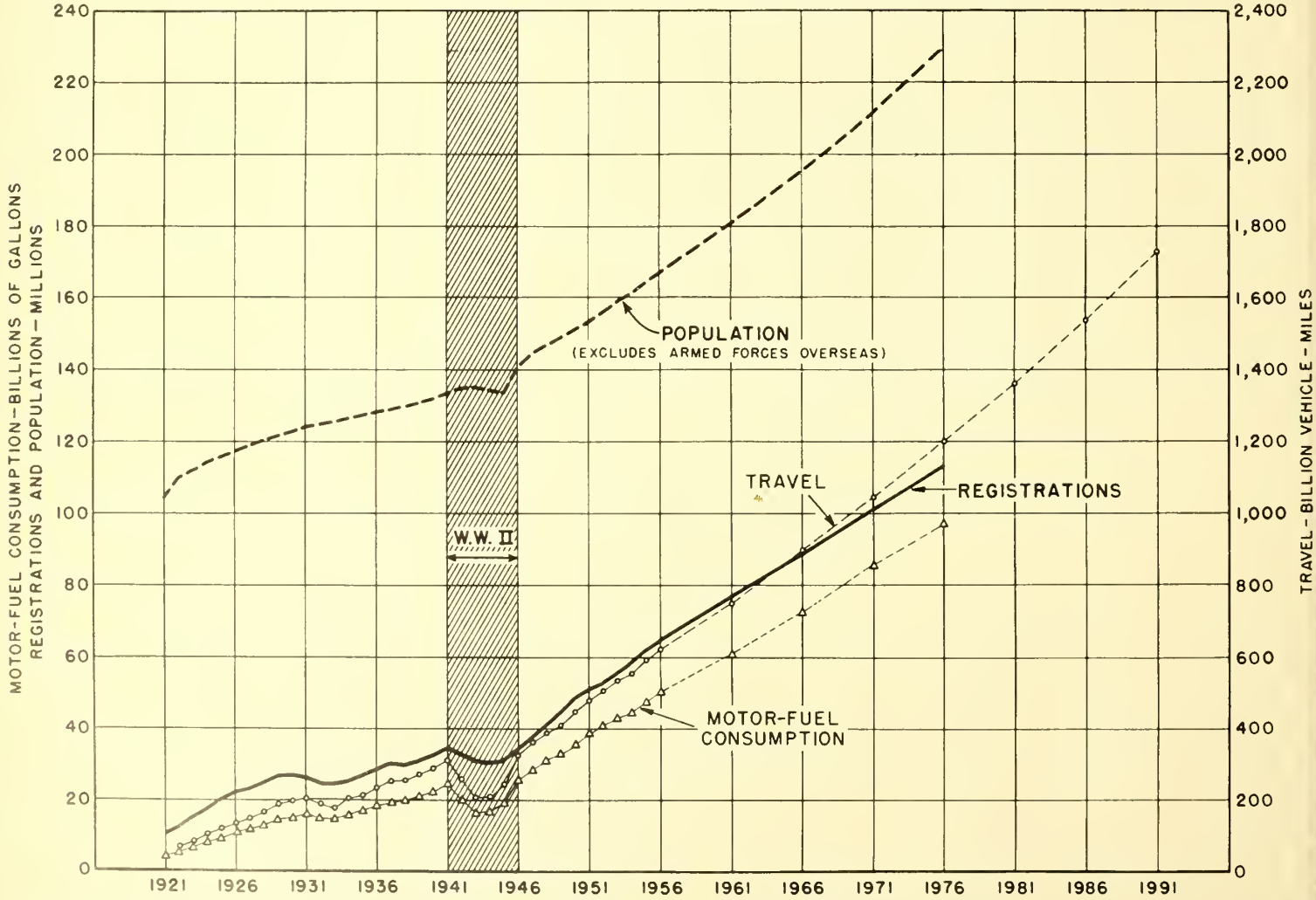

Figure 1.—State estimates of population, motor-vehicle registrations, travel, and motor-fuel consumption in the United States (excludes Alaska and Hawaii) for selected years.

Table 2.—State estimates of total population and population 15 to 74 years of age in the United States (excludes Alaska and Hawaii) by census division, 1956 and 1976

Census division	1956 population				1976 population							
	All ages		Driving age, 15–74		All ages				Driving age, 15–74			
	Persons	Percent of total	Persons	Percent of total	Persons	Percent of total	Ratio: 1976/1956	Numerical increase, 1956–76	Persons	Percent of total	Ratio: 1976/1956	Numerical increase, 1956–76
	Thousands		*Thousands*		*Thousands*			*Thousands*	*Thousands*			*Thousands*
New England	9,881	5.9	6,812	6.0	12,171	5.3	1.23	2,290	8,060	5.2	1.18	1,248
Middle Atlantic	32,669	19.5	22,849	20.2	41,231	17.9	1.26	8,562	28,139	18.2	1.23	5,290
South Atlantic (North)	9,701	5.8	6,673	5.9	12,560	5.5	1.29	2,859	8,401	5.5	1.26	1,728
South Atlantic (South)	14,326	8.6	9,443	8.3	21,346	9.3	1.49	7,020	14,099	9.1	1.49	4,656
East North Central	34,185	20.4	23,247	20.5	45,967	20.0	1.34	11,782	31,601	20.5	1.36	8,354
East South Central	11,833	7.1	7,715	6.8	13,251	5.8	1.12	1,418	8,777	5.7	1.14	1,062
West North Central	15,022	9.0	9,915	8.7	17,634	7.7	1.17	2,612	11,397	7.4	1.15	1,482
West South Central	16,017	9.6	10,457	9.2	21,469	9.3	1.34	5,452	14,020	9.1	1.34	3,563
Mountain	6,118	3.7	3,976	3.5	10,084	4.4	1.65	3,966	6,482	4.2	1.63	2,506
Pacific	17,498	10.4	12,333	10.9	34,045	14.8	1.95	16,547	23,344	15.1	1.89	11,011
All census divisions	167,250	100.0	113,420	100.0	229,758	100.0	1.37	62,508	154,320	100.0	1.36	40,900

The projections reported in table 2 show that the ratio of persons in the driver age group to the total population is expected to remain almost constant on a nationwide basis during the forecast period; two out of three persons in the total population are in the potential drivers group in both study years.

A comparison of relative change by census divisions shows that the largest gain in potential drivers is expected in the Pacific division. This gain is offset by losses anticipated in the Middle Atlantic, East South Central, and West North Central divisions.

Trends by States

The forecasts by the individual States (table 3) show that by 1976 the population is expected to more than double in California, Florida, and New Mexico. Three other States—Arizona, Nevada, and Utah—are expected to have increases of between 80 and 100 percent. No State expects to experience a net population decrease during the 20-year period, but the distribution of the percentage increases in population anticipated by the several States reveals that West Virginia and Kentucky expect only a 5-percent increase, and Arkansas and the District of Columbia expect increases of only 9 percent. The majority of the States (33) anticipate population increases ranging from 10 to 40 percent.

Motor-Vehicle Registrations

According to the forecasts prepared by the States, approximately 114 million motor vehicles will be using the Nation's highways in 1976. This forecast represents an increase of 49 million vehicles, or 75 percent, over 1956 registrations. Implicit in such a prediction is a fairly substantial increase in the density of motor-vehicle ownership. Figure 3 portrays the motor-vehicle registration projections by census divisions. Data for the two selected study years are compared in table 4, and detailed data by census divisions and States are shown in table 5.

It may be noted that the motor-vehicle registration figures for 1956 shown in this article differ slightly from those published in *Highway Statistics 1956*.[4] Although there are several reasons for the slight variation, the major one was the use of preliminary data for this study by many States.

Forecasts by census divisions

A comparison of the State motor-vehicle registration forecasts by census divisions (table 4) indicates that the 1976 registrations for the Pacific division will be more than double the 1956 figure, an increase of 9.8 million vehicles, or 116 percent. This anticipated registration increase, like the population forecast for this division, is the greatest found in any division. In the Mountain division an increase of 2.9 million motor vehicles is expected during the period which represents a 100-percent increase over 1956. In the South Atlantic (South) division the expected increase is 5.3 million vehicles, or 98 percent. The remaining divisions are expected to increase at a somewhat lower rate than the national average of 75 percent, with the West North Central division showing an increase of only 44 percent.

Registration forecasts by States

In the forecasts of motor-vehicle registrations by individual States (table 5), New Mexico anticipates the greatest percentage increase during the 20-year period, 180 percent, followed by Utah, Florida, and California. Iowa, West Virginia, and Nebraska predict the lowest State percentage increases; Iowa's increase for the 20-year period being only 22 percent. The District of Columbia anticipates an increase of only 15 percent.

Numerically, California expects to have 14.6 million registered motor vehicles in 1976, or 13 percent of the national total, as compared with the 6.5 million and 10 percent of the total in 1956. New York anticipates a registration total of 8.0 million while Pennsylvania, Ohio, and Texas each expects over 6 million motor vehicles to be registered in their States in 1976.

Density of motor-vehicle ownership

Table 6 shows the 1956 and 1976 State estimates of motor vehicles registered per 100 persons in the total population and the potential driver age group for each census division.

In 1951 there were 33.7 registered motor vehicles per 100 persons; by 1956 there were 38.9; and in 1976 there are expected to be 49.5 vehicles per 100 persons. The 1976 estimate

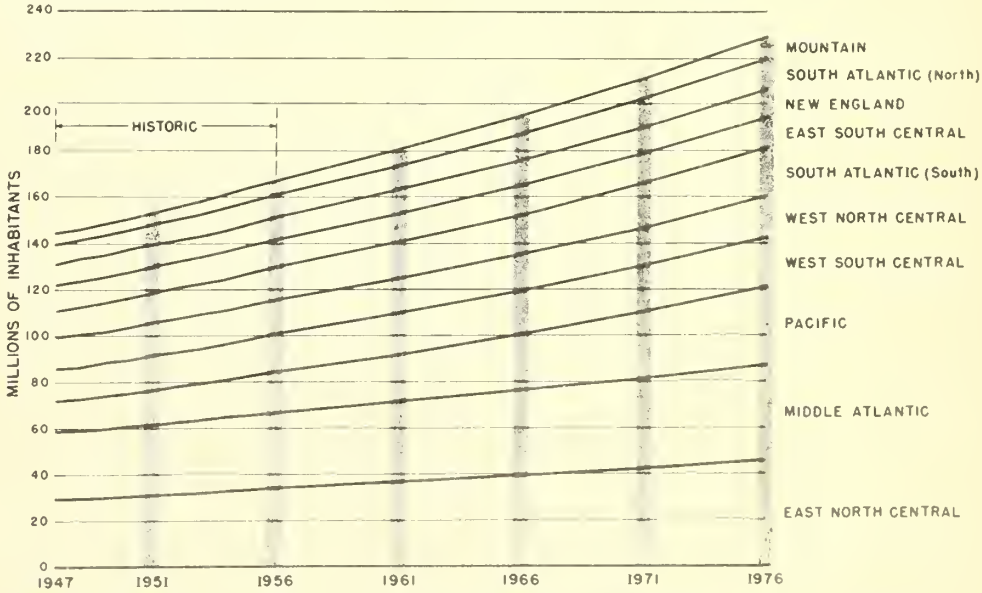

Figure 2.—State estimates of population in the United States by census divisions for selected years, 1947–76.

[4] *Highway Statistics 1956*, Bureau of Public Roads, table MV–1, p. 13.

of approximately 1 motor vehicle for every 2 persons in the total population, and 3 motor vehicles for every 4 persons in the driver age group, appears reasonable when the historic trend from 1951 to 1956 is considered. The density of motor-vehicle registrations for the historic period as well as the projected trend is as follows:

Year	Persons per vehicle
1951	2.97
1952	2.94
1953	2.83
1954	2.76
1955	2.63
1956	2.57
1961	2.35
1966	2.19
1971	2.09
1976	2.02

The Mountain division is expected to have the highest density of motor-vehicle ownership, 57.6 vehicles per 100 persons by 1976, and the Middle Atlantic and South Atlantic (North) divisions are expected to have the lowest, the values being 44.0 and 44.5.

As is to be expected, the census divisions having the highest density of motor-vehicle ownership in relation to total population also have the highest densities of motor-vehicle ownership in the driver age group. However, the divisions having the highest densities of ownership in 1956 are expected to experience the lowest percentage increase. On the other hand, in the East South Central division a 52-percent increase in motor vehicles registered per 100 persons during the 20-year period is anticipated, moving that division from the lowest of the 10 census divisions in ownership density in 1956 to fifth in 1976. This is the result of a combination of the lowest forecast of population increase (12 percent) and a relatively high (69 percent) projected increase in registrations. The Pacific division is expected to show the smallest percentage increase.

The 1976 Mountain division forecast of almost 9 motor vehicles for every 10 persons in the driver age group is the highest density forecast of all divisions. The West North Central and Pacific divisions are second and third with ratios of 82.4 and 77.9, respectively. The Middle Atlantic and South Atlantic (North) divisions, with 64.4 and 66.5 motor vehicles per 100 persons of driving age, respectively, rank lowest.

Table 3.—State forecasts of population in the United States by census division and State for selected years, 1956–76

Census division and State	1956		1961			1966			1971			1976		
	Total	Driving age group, 15–74	Total	Ratio: 1961/1956	Driving age group, 15–74	Total	Ratio: 1966/1956	Driving age group, 15–74	Total	Ratio: 1971/1956	Driving age group, 15–74	Total	Ratio: 1976/1956	Driving age group, 15–74
	Thousands	*Thousands*	*Thousands*		*Thousands*	*Thousands*		*Thousands*	*Thousands*		*Thousands*	*Thousands*		*Thousands*
United States	**167,250**	**113,420**	**180,656**	**1.08**	**121,041**	**195,353**	**1.17**	**131,360**	**211,653**	**1.27**	**142,858**	**229,758**	**1.37**	**154,320**
New England	**9,881**	**6,812**	**10,338**	**1.05**	**7,075**	**10,917**	**1.10**	**7,441**	**11,535**	**1.17**	**7,772**	**12,171**	**1.23**	**8,060**
Connecticut	2,313	1,619	2,429	1.05	1,712	2,627	1.14	1,806	2,853	1.23	1,900	3,114	1.35	1,994
Maine	934	644	968	1.04	668	1,002	1.07	691	1,036	1.11	715	1,070	1.15	738
Massachusetts	4,890	3,362	5,140	1.05	3,480	5,410	1.11	3,673	5,680	1.16	3,827	5,920	1.21	3,946
New Hampshire	558	376	586	1.05	391	615	1.10	415	650	1.16	439	692	1.24	459
Rhode Island	815	573	838	1.03	586	878	1.08	611	918	1.13	636	958	1.18	658
Vermont	371	238	377	1.02	238	385	1.04	245	398	1.07	255	417	1.12	265
Middle Atlantic	**32,669**	**22,849**	**34,554**	**1.06**	**23,874**	**36,413**	**1.11**	**25,022**	**38,641**	**1.18**	**26,520**	**41,231**	**1.26**	**28,139**
New Jersey	5,420	3,784	5,890	1.09	4,088	6,350	1.17	4,420	6,910	1.27	4,739	7,500	1.38	5,100
New York	16,256	11,487	17,203	1.06	12,025	18,136	1.12	12,500	19,244	1.18	13,325	20,533	1.26	14,300
Pennsylvania	10,993	7,578	11,461	1.04	7,761	11,927	1.08	8,102	12,487	1.14	8,456	13,198	1.20	8,739
South Atlantic (North)	**9,701**	**6,673**	**10,327**	**1.06**	**7,041**	**11,015**	**1.14**	**7,422**	**11,753**	**1.21**	**7,873**	**12,560**	**1.29**	**8,401**
Delaware	418	275	449	1.07	305	502	1.20	342	564	1.35	384	632	1.51	424
District of Columbia	844	626	864	1.02	618	884	1.05	622	903	1.07	634	923	1.09	635
Maryland	2,747	1,932	3,010	1.10	2,069	3,310	1.20	2,177	3,615	1.32	2,363	3,940	1.43	2,661
Virginia	3,635	2,522	3,914	1.08	2,720	4,205	1.16	2,916	4,537	1.25	3,110	4,912	1.35	3,303
West Virginia	2,057	1,318	2,090	1.02	1,329	2,114	1.03	1,365	2,134	1.04	1,382	2,153	1.05	1,378
South Atlantic (South)	**14,326**	**9,443**	**16,046**	**1.12**	**10,510**	**17,784**	**1.24**	**11,721**	**19,562**	**1.37**	**12,950**	**21,346**	**1.49**	**14,099**
Florida	3,885	2,774	4,885	1.26	3,488	5,885	1.51	4,202	6,885	1.77	4,916	7,885	2.03	5,630
Georgia	3,700	2,363	3,913	1.06	2,469	4,117	1.11	2,637	4,345	1.17	2,819	4,579	1.24	2,955
North Carolina	4,412	2,820	4,754	1.08	2,962	5,096	1.16	3,168	5,438	1.23	3,369	5,780	1.31	3,535
South Carolina	2,329	1,486	2,494	1.07	1,591	2,686	1.15	1,714	2,894	1.24	1,846	3,102	1.33	1,979
East North Central	**34,185**	**23,247**	**36,779**	**1.08**	**24,888**	**39,660**	**1.16**	**27,038**	**42,713**	**1.25**	**29,342**	**45,967**	**1.34**	**31,601**
Illinois	9,418	6,555	10,006	1.06	6,804	10,607	1.13	7,213	11,301	1.20	7,685	12,110	1.29	8,114
Indiana	4,418	2,953	4,858	1.10	3,243	5,298	1.20	3,533	5,738	1.30	3,823	6,178	1.40	4,143
Michigan	7,516	5,162	7,911	1.05	5,516	8,580	1.14	6,155	9,329	1.24	6,861	10,162	1.35	7,547
Ohio	9,064	6,089	9,957	1.10	6,727	10,850	1.20	7,365	11,742	1.30	8,003	12,635	1.39	8,641
Wisconsin	3,769	2,488	4,047	1.07	2,598	4,325	1.15	2,772	4,603	1.22	2,970	4,882	1.30	3,156
East South Central	**11,833**	**7,715**	**12,138**	**1.03**	**7,867**	**12,494**	**1.06**	**8,195**	**12,855**	**1.09**	**8,533**	**13,251**	**1.12**	**8,777**
Alabama	3,127	1,969	3,204	1.02	1,997	3,262	1.04	2,066	3,337	1.07	2,129	3,451	1.10	2,183
Kentucky	3,020	1,943	3,059	1.01	1,944	3,080	1.02	1,978	3,118	1.03	2,011	3,178	1.05	2,020
Mississippi	2,176	1,430	2,195	1.01	1,444	2,300	1.06	1,530	2,400	1.10	1,615	2,500	1.15	1,708
Tennessee	3,510	2,373	3,680	1.05	2,482	3,852	1.10	2,621	4,000	1.14	2,778	4,122	1.17	2,866
West North Central	**15,022**	**9,915**	**15,576**	**1.04**	**10,099**	**16,159**	**1.08**	**10,489**	**16,844**	**1.12**	**10,982**	**17,634**	**1.17**	**11,397**
Iowa	2,704	1,764	2,772	1.03	1,771	2,838	1.05	1,822	2,903	1.07	1,872	2,970	1.10	1,886
Kansas	2,090	1,392	2,190	1.05	1,432	2,293	1.10	1,492	2,417	1.16	1,586	2,571	1.23	1,674
Minnesota	3,246	2,104	3,391	1.04	2,170	3,558	1.10	2,284	3,755	1.16	2,426	3,971	1.22	2,570
Missouri	4,235	2,890	4,404	1.04	2,951	4,576	1.08	3,075	4,786	1.13	3,216	5,030	1.19	3,333
Nebraska	1,414	930	1,442	1.02	928	1,478	1.05	946	1,522	1.08	980	1,577	1.12	1,004
North Dakota	646	397	669	1.04	411	690	1.07	424	712	1.10	438	733	1.13	451
South Dakota	687	438	708	1.03	436	726	1.06	446	749	1.09	464	782	1.14	479
West South Central	**16,017**	**10,457**	**17,186**	**1.07**	**11,118**	**18,520**	**1.16**	**12,083**	**19,930**	**1.24**	**13,117**	**21,469**	**1.31**	**14,020**
Arkansas	1,815	1,143	1,857	1.02	1,175	1,899	1.05	1,196	1,941	1.07	1,219	1,983	1.09	1,241
Louisiana	2,976	1,880	3,186	1.07	1,982	3,398	1.14	2,140	3,641	1.22	2,327	3,888	1.31	2,455
Oklahoma	2,315	1,586	2,416	1.04	1,655	2,645	1.14	1,812	2,800	1.21	1,918	2,960	1.28	2,028
Texas	8,911	5,848	9,727	1.09	6,306	10,578	1.19	6,935	11,548	1.30	7,653	12,638	1.42	8,296
Mountain	**6,118**	**3,976**	**7,016**	**1.15**	**4,455**	**7,957**	**1.30**	**5,085**	**8,978**	**1.47**	**5,786**	**10,084**	**1.65**	**6,482**
Arizona	1,055	726	1,293	1.23	830	1,519	1.44	978	1,745	1.65	1,132	1,971	1.87	1,290
Colorado	1,579	1,052	1,741	1.10	1,144	1,912	1.21	1,266	2,108	1.34	1,406	2,327	1.47	1,536
Idaho	620	389	657	1.06	404	693	1.12	431	737	1.19	463	788	1.27	493
Montana	634	409	663	1.05	419	692	1.09	438	721	1.14	461	750	1.18	476
Nevada	245	172	298	1.22	206	350	1.43	244	408	1.67	284	474	1.93	328
New Mexico	844	517	1,058	1.25	642	1,301	1.54	803	1,559	1.85	983	1,834	2.17	1,155
Utah	825	507	967	1.17	595	1,129	1.37	694	1,312	1.59	807	1,521	1.84	935
Wyoming	316	204	339	1.07	215	361	1.14	231	388	1.23	250	419	1.33	269
Pacific	**17,498**	**12,333**	**20,696**	**1.18**	**14,114**	**24,434**	**1.40**	**16,864**	**28,842**	**1.65**	**19,983**	**34,045**	**1.95**	**23,344**
California	13,116	9,392	15,758	1.20	10,821	18,933	1.44	13,136	22,746	1.73	15,831	27,328	2.08	18,810
Oregon	1,730	1,160	1,957	1.13	1,305	2,195	1.27	1,483	2,420	1.40	1,642	2,636	1.52	1,774
Washington	2,652	1,781	2,981	1.12	1,988	3,306	1.25	2,245	3,676	1.39	2,510	4,081	1.54	2,760
Hawaii	523	364	566	1.08	398	590	1.13	420	613	1.17	442	629	1.20	459
Puerto Rico	2,441	-------	2,625	1.08	-------	2,807	1.15	-------	2,982	1.22	-------	3,146	1.29	-------
Grand Total	**170,214**	**113,784**	**183,847**	**1.08**	**121,439**	**198,750**	**1.17**	**131,780**	**215,248**	**1.26**	**143,300**	**233,533**	**1.37**	**154,779**

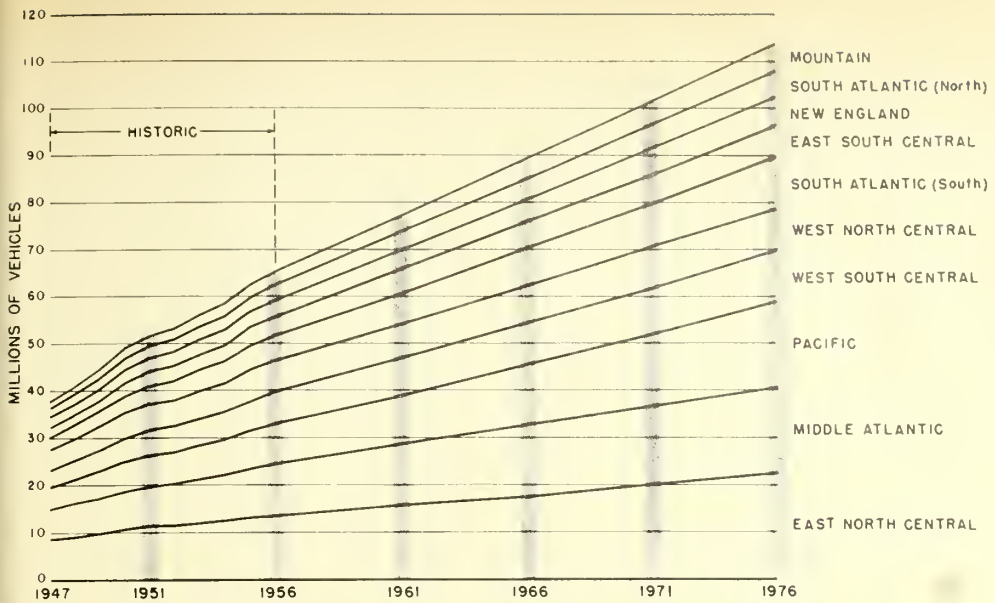

Figure 3.—State estimates of motor-vehicle registrations in the United States by census divisions for selected years, 1947–76.

Density of ownership generally tends to be greatest in the more rural States and least in those that are highly urbanized. Wyoming anticipates the highest density of motor-vehicle ownership in 1976, with 75 vehicles per hundred persons. Montana, with 70, is next highest, and is followed by Idaho, Kentucky, and Kansas. The lowest prediction among the States, reported by West Virginia, is 35 motor vehicles per hundred persons, and New York, with 39, is the second lowest.

The percentage increases in ownership densities for the 1956–76 period show that Massachusetts, in spite of its low ranking of 48 vehicles for every 100 persons, anticipates a substantial increse of 44 percent whereas Wyoming anticipates only a 35-percent increase. Kentucky, because of an extremely low forecast of population increase combined with a fairly high forecast of registrations, expects a 78-percent increase in ownership density. A similar situation exists in Alabama and Arkansas.

Population and registration gains compared

Very substantial gains in both population and registrations are expected in the Mountain and Pacific divisions during the forecast period. The same situation prevails to a somewhat lesser degree in the South Atlantic (South) division. This trend is in agreement with the growth in industrial and economic stature which these geographical areas have been experiencing in the past and are expected to experience in the future. The East North Central and Middle Atlantic divisions are expected to have the greatest number of inhabitants in 1976, but they will be seriously challenged by the Pacific division. In motor-vehicle registrations, the Pacific division is expected to equal the Middle Atlantic division and to be exceeded only by the East North Central division.

Trucks and buses

In 1956 the combined total of trucks and buses registered was reported to be 10.6 million which was 16.3 percent of the reported 65.1 million motor vehicles registered (table 5). For 1976, the estimated total of trucks and buses was 18.7 million. This number represents 16.4 percent of the 113.6 million motor vehicles expected to be registered in that year.

At first it would appear that the forecasters expected the ratio of trucks and buses to total registered vehicles to remain at about the same level for the 20-year period. Closer examination of the forecasts by census divisions and States reveals, however, that this nationwide relationship is only a coincidental one reflecting the combination of differing trends among the census divisions. The percentage of trucks and buses to total motor vehicles is expected to decline in 7 of the 10 geographic areas, while gains of 1.9 percentage points in the South Atlantic (North), 2.0 in the West North Central, and 4.1 in the Pacific division are indicated. The range of decreases, in contrast with the three divisions showing increases, is much lower. The East North Central and East South Central each estimated a 0.2-percentage point decrease from 1956 to 1976, while New England predicted the greatest decrease, 2.2 percentage points.

The reasons underlying these varying projected changes are not immediately evident. The forecasts by several States reflect continuation of recently observed trends, but whether the existing trends can be expected to continue throughout the 20-year period is, of course, conjectural. A regional pattern of truck and bus ownership was revealed and is expected to continue, with some exceptions, to 1976. Thus, the New England, Middle Atlantic, and East North Central divisions reported considerably lower levels of truck and bus registrations in 1956 than did any of the other divisions except the Pacific; these three divisions—which are, in general, the most urban—expect to have even less trucks proportionally in 1976 than they had in 1956. The three divisions in the Southeast showed little relative change in truck and bus registrations from 1956 to 1976. The three divisions exhibiting the highest proportion of truck and bus registrations—the West North Central, West South Central, and Mountain divisions—are expected to occupy the same position in 1976. Only the Pacific division shows a pronounced shift in position, from among the lowest in percentage of total trucks and buses in 1956, to somewhat above the national average in 1976.

In considering the relative position of trucks and buses to total registrations, it should be borne in mind that in no case was the number of such vehicles registered expected to decline. Even in the New England and Middle Atlantic States, where the greatest percentage decline in relation to total registrations of trucks and buses is forecast, the actual number registered is expected to increase by more than 40 percent.

Table 4.—State estimates of motor-vehicle registrations in the United States (excludes Alaska and Hawaii) by census division, 1956 and 1976

Census division	1956		1976		
	Total vehicle registrations	Percent of total	Total vehicle registrations	Percent of total	Ratio: 1976/1956
	Thousands		*Thousands*		
New England	3,598	5.5	6,076	5.3	1.69
Middle Atlantic	10,950	16.8	18,125	15.9	1.66
South Atlantic (North)	3,231	5.0	5,588	4.9	1.73
South Atlantic (South)	5,381	8.3	10,650	9.4	1.98
East North Central	13,547	20.8	22,236	19.6	1.64
East South Central	3,932	6.0	6,664	5.9	1.69
West North Central	6,516	10.0	9,386	8.3	1.44
West South Central	6,617	10.1	10,919	9.6	1.65
Mountain	2,908	4.5	5,807	5.1	2.00
Pacific	8,439	13.0	18,191	16.0	2.16
All census divisions	65,119	100.0	113,642	100.0	1.75

Motor-Vehicle Travel

Total motor-vehicle travel, as forecast by the States, is expected to reach an annual figure of 1.2 trillion vehicle-miles in 1976, representing an increase of 577 billion vehicle-miles, or 93 percent, over 1956. The travel trends, as predicted by the States, are somewhat higher than previous forecasts used in reports on the nation's highway needs, especially for the later years of the forecast period. Two of these reports [5] predicted that

[5] *A Ten-year National Highway Program*, A Report to the President, The President's Advisory Committee on a National Highway Program, January 1955, p. 6. *Needs of the Highway Systems, 1955–84*, House Document No. 120, Government Printing Office, March 1955.

81 million motor vehicles would travel 814 billion vehicle-miles in 1965. Estimates prepared for the Highway Cost Allocation Study indicate that in 1965 there will be 870 billion vehicle-miles traveled by 87 million motor vehicles. The differences in the forecasts become more apparent in the extended forecast period. For 1976, Highway Cost Allocation Study forecasts are 14 percent higher than the 1955 Nationwide Highway Finance Study; for 1991 they are 28 percent higher. The higher projection of traffic means, of course, that the highway needs will be greater. Increased travel is, however, only one of many factors influencing greater needs.

Historical data for 1947, 1951, and 1956, and forecasts of total travel for selected years through 1976 are shown by census divisions in figure 4, and a comparison of motor-vehicle travel for 1956 and 1976 is made in table 7. Detailed information for the individual States is contained in table 8.

Historically, total motor-vehicle travel has consistently increased at a more rapid rate than motor-vehicle registrations (table 1). This evidence was substantiated by the record of the consumption of motor fuel used on the highways.

The trend lines for highway travel and highway use of motor fuel have closely paralleled each other throughout the historical period, and have rather consistently run above the trends in motor-vehicle registrations. It seems reasonable to expect that total travel and motor-fuel consumption trends will continue in the near future, but that increases in the density of motor-vehicle ownership may be expected to put somewhat of a "brake" on their running ahead of the registration curve.

Forecasts by census divisions

A review of the travel forecasts by census divisions presented in table 7 and figure 4 shows that in the Pacific division a 149-percent increase during the forecast period is antie-

Table 5.—State forecasts of motor-vehicle registrations by census division and State for selected years, 1956–76

Census division and State	1956					1961						1966					
	Motor vehicles registered		Passenger cars	Trucks and buses		Motor vehicles registered			Passenger cars	Trucks and buses		Motor vehicles registered			Passenger cars	Trucks and buses	
	Total	Per 100 persons		Number	Percent of total	Total	Ratio: 1961/1956	Per 100 persons		Number	Percent of total	Total	Ratio: 1966/1956	Per 100 persons		Number	Percent of total
	Thousands		*Thousands*	*Thousands*		*Thousands*			*Thousands*	*Thousands*		*Thousands*			*Thousands*	*Thousands*	
United States	**65,119**	**38.9**	**54,535**	**10,584**	**16.3**	**77,002**	**1.18**	**42.6**	**64,405**	**12,597**	**16.4**	**89,161**	**1.37**	**45.6**	**74,640**	**14,521**	**16.3**
New England	**3,598**	**36.4**	**3,130**	**468**	**13.0**	**4,217**	**1.17**	**40.8**	**3,698**	**519**	**12.3**	**4,847**	**1.35**	**44.4**	**4,282**	**565**	**11.7**
Connecticut	955	41.3	842	113	11.8	1,155	1.21	47.6	1,027	128	11.1	1,355	1.42	51.6	1,212	143	10.6
Maine	340	36.4	270	70	20.6	375	1.10	38.7	300	75	20.0	410	1.21	40.9	328	82	20.0
Massachusetts	1,619	33.1	1,429	190	11.7	1,908	1.18	37.1	1,698	210	11.0	2,210	1.37	40.9	1,984	226	10.2
New Hampshire	225	40.3	184	41	18.2	266	1.18	45.4	217	49	18.4	302	1.34	49.1	248	54	17.9
Rhode Island	318	39.0	280	38	11.9	355	1.12	42.4	314	41	11.5	396	1.25	45.1	352	44	11.1
Vermont	141	38.0	125	16	11.3	158	1.12	41.9	142	16	10.1	174	1.23	45.2	158	16	9.2
Middle Atlantic	**10,950**	**33.5**	**9,561**	**1,389**	**12.7**	**12,965**	**1.18**	**37.5**	**11,361**	**1,604**	**12.4**	**14,805**	**1.35**	**40.7**	**13,057**	**1,748**	**11.8**
New Jersey	2,250	41.5	1,937	313	13.9	2,670	1.19	45.3	2,296	374	14.0	3,020	1.34	47.6	2,597	423	14.0
New York	4,810	29.6	4,279	531	11.0	5,725	1.19	33.3	5,150	575	10.0	6,475	1.35	35.7	5,875	600	9.3
Pennsylvania	3,890	35.4	3,345	545	14.0	4,570	1.17	39.9	3,915	655	14.3	5,310	1.37	44.5	4,585	725	13.7
South Atlantic (North)	**3,231**	**33.3**	**2,694**	**537**	**16.6**	**3,784**	**1.17**	**36.6**	**3,120**	**664**	**17.5**	**4,408**	**1.36**	**40.0**	**3,620**	**788**	**17.9**
Delaware	164	39.2	127	37	22.6	209	1.27	46.5	163	46	22.0	252	1.54	50.2	198	54	21.4
District of Columbia	198	23.5	174	24	12.1	200	1.01	23.1	176	24	12.0	210	1.06	23.8	185	25	11.9
Maryland	984	35.8	849	135	13.7	1,134	1.15	37.7	981	153	13.5	1,349	1.37	40.8	1,167	182	13.5
Virginia	1,315	36.2	1,091	224	17.0	1,604	1.22	41.0	1,324	280	17.5	1,910	1.45	45.4	1,561	349	18.3
West Virginia	570	27.7	453	117	20.5	637	1.12	30.5	476	161	25.3	687	1.21	32.5	509	178	25.9
South Atlantic (South)	**5,381**	**37.6**	**4,406**	**975**	**18.1**	**6,733**	**1.25**	**42.0**	**5,524**	**1,209**	**18.0**	**8,075**	**1.50**	**45.4**	**6,655**	**1,420**	**17.6**
Florida	1,783	45.9	1,517	266	14.9	2,408	1.35	49.3	2,047	361	15.0	3,033	1.70	51.5	2,578	455	15.0
Georgia	1,273	34.4	1,017	256	20.1	1,579	1.24	40.4	1,251	328	20.8	1,868	1.47	45.4	1,494	374	20.0
North Carolina	1,516	34.4	1,212	304	20.1	1,769	1.17	37.2	1,415	354	20.0	2,044	1.35	40.1	1,635	409	20.0
South Carolina	809	34.7	660	149	18.4	977	1.21	39.2	811	166	17.0	1,130	1.40	42.1	948	182	16.1
East North Central	**13,547**	**39.6**	**11,711**	**1,836**	**13.6**	**15,581**	**1.15**	**42.4**	**13,439**	**2,142**	**13.7**	**17,775**	**1.31**	**44.8**	**15,361**	**2,414**	**13.6**
Illinois	3,408	36.2	2,984	424	12.4	3,960	1.16	39.6	3,479	481	12.1	4,500	1.32	42.4	3,979	521	11.6
Indiana	1,849	41.9	1,516	333	18.0	2,174	1.18	44.8	1,783	391	18.0	2,499	1.35	47.2	2,049	450	18.0
Michigan	3,138	41.8	2,747	391	12.5	3,439	1.10	43.5	2,992	447	13.0	3,837	1.22	44.7	3,338	499	13.0
Ohio	3,706	40.9	3,271	435	11.7	4,309	1.16	43.3	3,793	516	12.0	4,986	1.35	46.0	4,389	597	12.0
Wisconsin	1,446	38.4	1,193	253	17.5	1,699	1.17	42.0	1,392	307	18.1	1,953	1.35	45.2	1,606	347	17.8
East South Central	**3,932**	**33.2**	**3,229**	**703**	**17.9**	**4,777**	**1.21**	**39.4**	**3,923**	**854**	**17.9**	**5,514**	**1.40**	**44.1**	**4,537**	**977**	**17.7**
Alabama	1,084	34.7	873	211	19.5	1,313	1.21	41.0	1,064	249	19.0	1,517	1.40	46.5	1,229	288	19.0
Kentucky	1,061	35.1	848	213	20.1	1,344	1.27	43.9	1,081	263	19.6	1,614	1.52	52.4	1,304	310	19.2
Mississippi	662	30.4	600	62	9.4	775	1.17	35.3	708	67	8.6	860	1.30	37.4	794	66	7.7
Tennessee	1,125	32.1	908	217	19.3	1,345	1.20	36.5	1,070	275	20.4	1,523	1.35	39.5	1,210	313	20.6
West North Central	**6,516**	**43.4**	**5,140**	**1,376**	**21.1**	**7,267**	**1.12**	**46.7**	**5,679**	**1,588**	**21.9**	**8,012**	**1.23**	**49.6**	**6,230**	**1,782**	**22.2**
Iowa	1,201	44.4	975	226	18.8	1,287	1.07	46.4	1,035	252	19.6	1,364	1.14	48.1	1,092	272	19.9
Kansas	1,066	51.0	812	254	23.8	1,196	1.12	54.6	911	285	23.8	1,327	1.24	57.9	1,011	316	23.8
Minnesota	1,411	43.5	1,163	248	17.6	1,631	1.16	48.1	1,337	294	18.0	1,850	1.31	52.0	1,513	337	18.2
Missouri	1,544	36.5	1,235	309	20.0	1,717	1.11	39.0	1,348	369	21.5	1,910	1.24	41.7	1,484	426	22.3
Nebraska	659	46.6	502	157	23.8	729	1.11	50.6	550	179	24.6	785	1.19	53.1	589	196	25.0
North Dakota	307	47.5	210	97	31.6	344	1.12	51.4	232	112	32.6	379	1.23	54.9	252	127	33.5
South Dakota	328	47.7	243	85	25.9	363	1.11	51.3	266	97	26.7	397	1.21	54.7	289	108	27.2
West South Central	**6,617**	**41.3**	**5,124**	**1,493**	**22.6**	**7,757**	**1.17**	**45.1**	**6,030**	**1,727**	**22.3**	**8,914**	**1.35**	**48.1**	**6,955**	**1,959**	**22.0**
Arkansas	608	33.5	420	188	30.9	663	1.09	35.7	455	208	31.4	760	1.25	40.0	516	244	32.1
Louisiana	1,006	33.8	795	211	21.0	1,249	1.24	39.2	994	255	20.4	1,493	1.48	43.9	1,194	299	20.0
Oklahoma	1,055	45.6	782	273	25.9	1,210	1.15	50.1	896	314	26.0	1,425	1.35	53.9	1,056	369	25.9
Texas	3,948	44.3	3,127	821	20.8	4,635	1.17	47.7	3,685	950	20.5	5,236	1.33	49.5	4,189	1,047	20.0
Mountain	**2,908**	**47.5**	**2,192**	**716**	**24.6**	**3,589**	**1.23**	**51.2**	**2,711**	**878**	**24.5**	**4,295**	**1.48**	**54.0**	**3,254**	**1,041**	**24.2**
Arizona	432	40.9	324	108	25.0	564	1.31	43.6	420	144	25.5	697	1.61	45.9	515	182	26.1
Colorado	770	48.8	600	170	22.1	882	1.15	50.7	688	194	22.0	1,006	1.31	52.6	785	221	22.0
Idaho	345	55.6	256	89	25.8	417	1.21	63.5	309	108	25.9	475	1.38	68.5	352	123	25.9
Montana	347	54.7	239	108	31.1	386	1.11	58.2	266	120	31.1	432	1.24	62.4	298	134	31.0
Nevada	128	52.2	99	29	22.7	167	1.30	56.0	129	38	22.8	205	1.60	58.6	159	46	22.4
New Mexico	348	41.2	260	88	25.3	485	1.39	45.8	369	116	23.9	625	1.80	48.0	481	144	23.0
Utah	362	43.9	294	68	18.8	473	1.31	48.9	383	90	19.0	607	1.68	53.8	492	115	18.9
Wyoming	176	55.7	120	56	31.8	215	1.22	63.4	147	68	31.6	248	1.41	68.7	172	76	30.6
Pacific	**8,439**	**48.2**	**7,348**	**1,091**	**12.9**	**10,332**	**1.22**	**49.9**	**8,920**	**1,412**	**13.7**	**12,516**	**1.48**	**51.2**	**10,689**	**1,827**	**14.6**
California	6,452	49.2	5,652	800	12.4	7,929	1.23	50.3	6,869	1,060	13.4	9,708	1.50	51.3	8,293	1,415	14.6
Oregon	808	46.7	730	78	9.7	989	1.22	50.5	895	94	9.5	1,168	1.45	53.2	1,055	113	9.7
Washington	1,179	44.5	966	213	18.1	1,414	1.20	47.4	1,156	258	18.2	1,640	1.39	49.6	1,341	299	18.2
Hawaii	188	35.9	160	28	14.9	218	1.16	38.5	185	33	15.1	238	1.27	40.3	201	37	15.5
Puerto Rico	123	5.0	80	43	35.0	175	1.42	6.7	122	53	30.3	196	1.59	7.0	147	49	25.0
Grand total	65,430	38.4	54,775	10,655	16.3	77,395	1.18	42.1	64,712	12,683	16.4	89,595	1.37	45.1	74,988	14,607	16.3

ipated. An increase of 118 percent is expected in the Mountain division. The South Atlantic (North) and South Atlantic (South) divisions are the only other divisions with percentage increases above the national average of 93 percent. The lowest increase, 65 percent, was reported in the West North Central division.

In spite of the wide variations in expected rates of increase, the Nation's overall travel pattern with respect to geographic areas is not expected to change greatly in the next 20 years. Thus, the New England, Middle Atlantic, and East North Central divisions, which accounted for 43.1 percent of total travel in 1956, are expected to account for 40.4 percent in 1976. The South Atlantic (North), South Atlantic (South), and East South Central divisions are expected to account for exactly the same proportion of total travel, 20.2 percent, in 1976 as was estimated for 1956. The West North Central and West South Central divisions, where population and registration increases are expected to be rather moderate, are predicted to account for only 17.8 percent of total travel in 1976 as compared to the 1956 estimate of 19.4 percent. Finally, the Mountain and Pacific divisions are expected to have 21.6 percent of the 1976 total travel, representing a 25-percent increase over the 1956 estimate of 17.3 percent.

Travel forecasts by States

Nevada, with an anticipated increase in total travel of 188 percent, has the highest relative forecast for any State. California and New Mexico are next highest with 165 percent each, followed closely by Utah with 162 percent. Maine anticipates the lowest percentage increase, 39 percent, with West Virginia and Vermont having the next two lowest (table 8).

Average travel per vehicle

The percentage increase in total travel as predicted by the States for the 1956–76 period was 93 percent (table 7). This percentage,

Table 5.—State forecasts of motor-vehicle registrations by census division and State for selected years, 1956–76—(*Continued*)

Census division and State	1971						1976					
	Motor vehicles registered			Passenger cars	Trucks and buses		Motor vehicles registered			Passenger cars	Trucks and buses	
	Total	Ratio: 1971/1956	Per 100 persons		Number	Percent of total	Total	Ratio: 1976/1956	Per 100 persons		Number	Percent of total
	Thousands			*Thousands*	*Thousands*		*Thousands*			*Thousands*	*Thousands*	
United States	**101,240**	**1.55**	**47.8**	**84,716**	**16,524**	**16.3**	**113,642**	**1.75**	**49.5**	**94,958**	**18,684**	**16.4**
New England	**5,465**	**1.52**	**47.4**	**4,853**	**612**	**11.2**	**6,076**	**1.69**	**49.9**	**5,419**	**657**	**10.8**
Connecticut	1,555	1.63	54.5	1,397	158	10.2	1,755	1.84	56.4	1,582	173	9.9
Maine	443	1.30	42.8	354	89	20.1	476	1.40	44.5	381	95	20.0
Massachusetts	2,510	1.55	44.2	2,267	243	9.7	2,810	1.74	47.5	2,551	259	9.2
New Hampshire	332	1.48	51.1	272	60	18.1	360	1.60	52.0	295	65	18.1
Rhode Island	437	1.37	47.6	391	46	10.5	477	1.50	49.8	428	49	10.3
Vermont	188	1.33	47.2	172	16	8.5	198	1.40	47.5	182	16	8.1
Middle Atlantic	**16,469**	**1.50**	**42.6**	**14,596**	**1,873**	**11.4**	**18,125**	**1.66**	**44.0**	**16,145**	**1,980**	**10.9**
New Jersey	3,380	1.50	48.9	2,907	473	14.0	3,750	1.67	50.0	3,225	525	14.0
New York	7,169	1.49	37.3	6,550	619	8.6	7,985	1.66	38.9	7,350	635	8.0
Pennsylvania	5,920	1.52	47.4	5,139	781	13.2	6,390	1.64	48.4	5,570	820	12.8
South Atlantic (North)	**5,020**	**1.55**	**42.7**	**4,107**	**913**	**18.2**	**5,588**	**1.73**	**44.5**	**4,552**	**1,036**	**18.5**
Delaware	296	1.80	52.5	234	62	20.9	340	2.07	53.8	269	71	20.9
District of Columbia	219	1.11	24.3	193	26	11.9	228	1.15	24.7	201	27	11.8
Maryland	1,587	1.61	43.9	1,374	213	13.4	1,809	1.84	45.9	1,566	243	13.4
Virginia	2,190	1.67	48.3	1,771	419	19.1	2,456	1.87	50.0	1,965	491	20.0
West Virginia	728	1.28	34.1	535	193	26.5	755	1.32	35.1	551	204	27.0
South Atlantic (South)	**9,386**	**1.74**	**48.0**	**7,746**	**1,640**	**17.5**	**10,650**	**1.98**	**49.9**	**8,798**	**1,852**	**17.4**
Florida	3,658	2.05	53.1	3,109	549	15.0	4,283	2.40	54.3	3,640	643	15.0
Georgia	2,157	1.69	49.6	1,725	432	20.0	2,439	1.92	53.3	1,952	487	20.0
North Carolina	2,306	1.52	42.4	1,845	461	20.0	2,543	1.68	44.0	2,034	509	20.0
South Carolina	1,265	1.56	43.7	1,067	198	15.7	1,385	1.71	44.6	1,172	213	15.4
East North Central	**20,001**	**1.48**	**46.8**	**17,315**	**2,686**	**13.4**	**22,236**	**1.64**	**48.1**	**19,257**	**2,979**	**13.4**
Illinois	5,050	1.48	44.7	4,490	560	11.1	5,581	1.64	46.1	4,967	614	11.0
Indiana	2,824	1.53	49.2	2,316	508	18.0	3,149	1.70	51.0	2,582	567	18.0
Michigan	4,257	1.36	45.6	3,704	553	13.0	4,706	1.50	46.3	4,094	612	13.0
Ohio	5,663	1.53	48.2	4,985	678	12.0	6,340	1.71	50.2	5,581	759	12.0
Wisconsin	2,207	1.53	47.9	1,820	387	17.5	2,460	1.70	50.4	2,033	427	17.4
East South Central	**6,153**	**1.56**	**47.9**	**5,064**	**1,089**	**17.7**	**6,664**	**1.69**	**50.3**	**5,484**	**1,180**	**17.7**
Alabama	1,738	1.60	52.1	1,408	330	19.0	1,939	1.79	56.2	1,571	368	19.0
Kentucky	1,825	1.72	58.5	1,479	346	19.0	1,985	1.87	62.5	1,611	374	18.8
Mississippi	940	1.42	39.2	867	73	7.8	1,010	1.53	40.4	932	78	7.7
Tennessee	1,650	1.47	41.3	1,310	340	20.6	1,730	1.54	42.0	1,370	360	20.8
West North Central	**8,710**	**1.34**	**51.7**	**6,736**	**1,974**	**22.7**	**9,386**	**1.44**	**53.2**	**7,222**	**2,164**	**23.1**
Iowa	1,424	1.19	49.1	1,134	290	20.4	1,471	1.22	49.5	1,165	306	20.8
Kansas	1,462	1.37	60.5	1,114	348	23.8	1,597	1.50	62.1	1,217	380	23.8
Minnesota	2,044	1.45	54.4	1,668	376	18.4	2,232	1.58	56.2	1,817	415	18.6
Missouri	2,088	1.35	43.6	1,606	482	23.1	2,249	1.46	44.7	1,716	533	23.7
Nebraska	842	1.28	55.3	627	215	25.5	905	1.37	57.4	670	235	26.0
North Dakota	414	1.35	58.1	272	142	34.3	449	1.46	61.3	292	157	35.0
South Dakota	436	1.33	58.2	315	121	27.8	483	1.47	61.8	345	138	28.6
West South Central	**9,911**	**1.50**	**49.7**	**7,756**	**2,155**	**21.7**	**10,919**	**1.65**	**50.9**	**8,548**	**2,371**	**21.7**
Arkansas	844	1.39	43.5	568	276	32.7	1,044	1.72	52.6	696	348	33.3
Louisiana	1,736	1.73	47.7	1,394	342	19.7	1,980	1.97	50.9	1,593	387	19.5
Oklahoma	1,578	1.50	56.4	1,169	409	25.9	1,693	1.60	57.2	1,254	439	25.9
Texas	5,753	1.46	49.8	4,625	1,128	19.6	6,202	1.57	49.1	5,005	1,197	19.3
Mountain	**5,024**	**1.73**	**56.0**	**3,819**	**1,205**	**24.0**	**5,807**	**2.00**	**57.6**	**4,428**	**1,379**	**23.7**
Arizona	829	1.92	47.5	611	218	26.3	962	2.23	48.8	707	255	26.5
Colorado	1,155	1.50	54.8	901	254	22.0	1,330	1.73	57.2	1,037	293	22.0
Idaho	503	1.46	68.2	374	129	25.6	538	1.56	68.3	400	138	25.7
Montana	479	1.38	66.4	331	148	30.9	525	1.51	70.0	362	163	31.0
Nevada	244	1.91	59.8	189	55	22.5	283	2.21	59.7	220	63	22.3
New Mexico	787	2.26	50.5	614	173	22.0	976	2.80	53.2	771	205	21.0
Utah	745	2.06	56.8	603	142	19.1	878	2.43	57.7	711	167	19.0
Wyoming	282	1.60	72.7	196	86	30.5	315	1.79	75.2	220	95	30.2
Pacific	**15,101**	**1.79**	**52.4**	**12,724**	**2,377**	**15.7**	**18,191**	**2.16**	**53.1**	**15,105**	**3,086**	**17.0**
California	11,886	1.84	52.3	9,986	1,900	16.0	14,550	2.26	53.2	12,005	2,545	17.5
Oregon	1,339	1.66	55.3	1,205	134	10.0	1,503	1.86	57.0	1,353	150	10.0
Washington	1,876	1.59	51.0	1,533	343	18.3	2,138	1.81	52.4	1,747	391	18.3
Hawaii	260	1.38	42.4	219	41	15.8	277	1.47	44.0	234	43	15.5
Puerto Rico	209	1.70	7.0	157	52	24.9	220	1.79	7.0	165	55	25.0
Grand total	**101,709**	**1.55**	**47.3**	**85,092**	**16,617**	**16.3**	**114,139**	**1.74**	**48.9**	**95,357**	**18,782**	**16.5**

Table 6.—State estimates of motor vehicles registered per 100 persons of all ages and of driving age 15–74, by census division, 1956 and 1976

Census division	Motor vehicles per 100 persons, total population		Percentage increase	Motor vehicles per 100 persons, age group 15–74		Percentage increase
	1956	1976		1956	1976	
New England	36.4	49.9	37.1	52.8	75.4	42.8
Middle Atlantic	33.5	44.0	31.3	47.9	64.4	34.4
South Atlantic (North)	33.3	44.5	33.6	48.4	66.5	37.4
South Atlantic (South)	37.6	49.9	32.7	57.0	75.5	32.5
East North Central	39.6	48.4	22.2	58.3	72.7	24.7
East South Central	33.2	50.3	51.5	51.0	75.9	48.8
West North Central	43.4	53.2	22.6	65.7	82.4	25.4
West South Central	41.3	50.9	23.2	63.3	77.9	23.1
Mountain	47.5	57.6	21.3	73.1	89.6	22.6
Pacific	48.2	53.4	10.8	68.4	77.9	13.9
All census divisions	38.9	49.5	27.2	57.4	73.6	28.2

substantially higher than the anticipated 75-percent increase in registrations, implies an increase in the average annual travel per registered motor vehicle. The 1956 average annual travel per vehicle, derived by dividing total travel by total registrations, was estimated to be 9,566 miles; the average is expected to be 10,562 by 1976, an increase of 10 percent. Although this is a relatively small percentage increase there are many who have doubts as to the validity of such a forecast. Such doubts are based largely on the belief that a family owning one vehicle and driving it 10,000 miles per year will not, on becoming a "two-car" or "car-and-truck" family, drive each vehicle 10,000 miles per year. Although this consideration is a valid one, there are several other factors which may have considerable weight in determining future rates of travel per vehicle. Among these factors are the expected continuing accelerated development of suburban areas; the development and expansion of the highway transportation industry; the anticipated growth in the Nation's economy, wealth, and population; and increased leisure time brought about by great increases in per capita productivity.

An examination of the State forecasts of average annual travel per registered vehicle shows a wide variation, not only for the two study years, but also among the States. The travel per registered vehicle—which is derived from the total travel of all motor vehicles, resident and nonresident, expected within the State, divided by total vehicle registrations of the State—is definitely affected by the State's geographic size and its location in connection with the major traffic streams of the nation. A State through which a major traffic corridor passes may be expected to show a rather high average travel per registered vehicle. To some extent, the States having special attractions for tourists will show similar travel patterns. Other factors, such as the percentage of trucks and buses to total registrations, will also have an appreciable affect on travel averages for the individual States. On a census division or national basis, however, the figures given may be considered entirely reasonable.

In 1956, the South Atlantic (North) division had the highest average annual travel per registered motor vehicle, 10,874 miles, followed by the South Atlantic (South) division with 10,081 miles. The lowest annual travel, 9,085 miles per registered vehicle was in the West North Central division. The 1976 projections show the South Atlantic (North) division as still the highest, with an estimated average annual travel of 12,442 miles per registered vehicle. Second highest, with 10,887 miles, was the Pacific census division. The lowest average annual travel, 9,800 miles per vehicle, was anticipated in the East South Central division, and the next lowest was the New England division estimate of 10,133 miles.

The annual average travel per registered motor vehicle points up, probably more than any other single item, the variations of the State forecasts. For example, the South Atlantic (South) division ranked second in 1956 but is expected to rank eighth in 1976; only a 1-percent increase is anticipated over the 20-year period, the smallest increase for any of the census divisions. The Pacific division, having the greatest annual travel rate increase, is expected to rise from seventh ranking to second in 1976. The West South Central division, ninth in 1956, is predicted to be third in 1976, and the New England division is expected to drop from fifth to ninth place.

For 1956, Virginia had the highest average annual travel per registered motor vehicle, with an estimate of 11,802 miles. New Mexico was next highest with 11,710 miles, followed by Georgia with 11,703 miles per vehicle. Montana reported the lowest annual travel, 7,660 miles per registered motor vehicle, and North Dakota's estimate of 7,697 miles was second lowest.

For 1976, the three highest State estimates were those for Nevada, Maryland, and New Jersey, ranging from 14,611 to 12,907 miles per registered vehicle. Louisiana's average vehicle travel in 1976 of 8,406 miles was the lowest among the States, and the next lowest estimates were for Montana (8,741) and North Dakota (8,820).

As noted in the discussion of the divisions, there appear to be some variations in the trends of average annual travel among the States. For example, the travel forecasts for

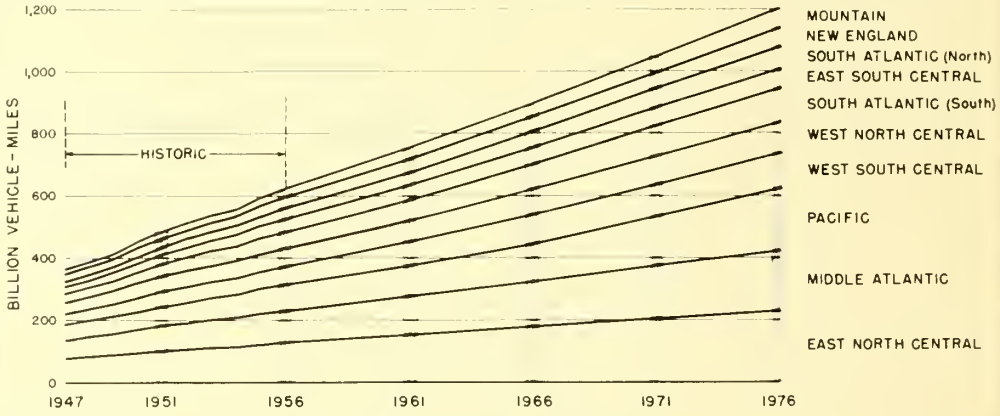

Figure 4.—State estimates of motor-vehicle travel in the United States by census divisions for selected years, 1947–76.

Table 7.—State estimates of total motor-vehicle travel in the United States (excludes Alaska and Hawaii) by census division, 1956 and 1976

Census division	1956			1976				
	Total travel, vehicle-miles	Percent of total	Travel per registered vehicle	Total travel, vehicle-miles	Percent of total	Travel per registered vehicle	Ratio: total travel 1976/1956	Ratio: travel per registered vehicle 1976/1956
	Millions		*Miles*	*Millions*		*Miles*		
New England	34,375	5.5	9,554	61,569	5.1	10,133	1.79	1.06
Middle Atlantic	103,637	16.7	9,465	191,513	16.0	10,566	1.85	1.12
South Atlantic (North)	35,134	5.6	10,874	69,528	5.8	12,442	1.98	1.14
South Atlantic (South)	54,248	8.7	10,081	108,521	9.0	10,190	2.00	1.01
East North Central	130,170	20.9	9,609	231,025	19.3	10,390	1.77	1.08
East South Central	36,979	5.9	9,405	65,306	5.4	9,800	1.77	1.04
West North Central	59,197	9.5	9,085	97,479	8.1	10,386	1.65	1.14
West South Central	61,762	9.9	9,334	116,610	9.7	10,680	1.89	1.14
Mountain	27,808	4.5	9,563	60,667	5.1	10,447	2.18	1.09
Pacific	79,622	12.8	9,435	198,045	16.5	10,887	2.49	1.15
All census divisions	622,932	100.0	9,566	1,200,263	100.0	10,562	1.93	1.10

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Table 8.—State forecasts of travel in the United States by census division and State for selected years, 1947–76

Census division and State	1947 travel			1951 travel			1956 travel		1961 travel			1966 travel			1971 travel			1976 travel		
	Total vehicle-miles	Ratio: 1947/1956	Per registered vehicle	Total vehicle-miles	Ratio: 1951/1956	Per registered vehicle	Total vehicle-miles	Per registered vehicle	Total vehicle-miles	Ratio: 1961/1956	Per registered vehicle	Total vehicle-miles	Ratio: 1966/1956	Per registered vehicle	Total vehicle-miles	Ratio: 1971/1956	Per registered vehicle	Total vehicle-miles	Ratio: 1976/1956	Per registered vehicle
	Millions		*Miles*	*Millions*		*Miles*	*Millions*	*Miles*	*Millions*		*Miles*	*Millions*		*Miles*	*Millions*		*Miles*	*Millions*		*Miles*
United States	**360,689**	**0.58**	**9,566**	**484,582**	**0.78**	**9,383**	**622,932**	**9,566**	**753,764**	**1.21**	**9,789**	**898,691**	**1.44**	**10,079**	**1,051,412**	**1.69**	**10,385**	**1,200,263**	**1.93**	**10,562**
New England	**21,682**	**.63**	**9,215**	**27,031**	**.79**	**9,156**	**34,375**	**9,554**	**41,359**	**1.20**	**9,808**	**48,299**	**1.41**	**9,965**	**55,088**	**1.60**	**10,080**	**61,569**	**1.79**	**10,133**
Connecticut	5,262	.59	8,829	6,795	.76	8,882	8,912	9,332	11,080	1.24	9,593	13,200	1.48	9,742	15,320	1.72	9,852	17,440	1.96	9,937
Maine	2,492	.67	10,383	2,993	.81	10,844	3,710	10,912	4,077	1.10	10,872	4,441	1.20	10,832	4,806	1.30	10,849	5,170	1.39	10,861
Massachusetts	9,660	.65	9,226	11,900	.80	8,834	14,896	9,201	18,100	1.22	9,486	21,400	1.44	9,683	24,700	1.66	9,841	27,800	1.87	9,893
New Hampshire	1,413	.56	9,235	1,938	.77	10,767	2,522	11,209	3,098	1.23	11,647	3,623	1.44	11,997	4,101	1.63	12,352	4,519	1.79	12,553
Rhode Island	1,830	.63	8,756	2,170	.75	8,314	2,910	9,151	3,370	1.16	9,493	3,810	1.31	9,621	4,160	1.43	9,519	4,490	1.54	9,413
Vermont	1,025	.72	9,491	1,235	.87	10,041	1,425	10,106	1,634	1.15	10,342	1,825	1.28	10,489	2,001	1.40	10,644	2,150	1.51	10,859
Middle Atlantic	**60,002**	**.58**	**9,286**	**80,269**	**.77**	**9,208**	**103,637**	**9,465**	**126,880**	**1.22**	**9,786**	**148,138**	**1.43**	**10,006**	**169,648**	**1.64**	**10,301**	**191,513**	**1.85**	**10,566**
New Jersey	12,710	.53	10,152	17,320	.72	10,200	24,000	10,667	30,500	1.27	11,423	35,900	1.50	11,887	41,900	1.75	12,396	48,400	2.02	12,907
New York	25,881	.62	8,983	33,562	.80	8,763	42,042	8,741	52,050	1.24	9,092	61,928	1.47	9,564	72,238	1.72	10,076	83,183	1.98	10,417
Pennsylvania	22,011	.59	9,198	29,387	.78	9,215	37,595	9,665	44,330	1.18	9,700	50,310	1.34	9,475	55,510	1.48	9,377	59,930	1.59	9,379
South Atlantic (North)	**17,192**	**.49**	**9,354**	**25,241**	**.72**	**10,084**	**35,134**	**10,874**	**43,084**	**1.23**	**11,386**	**51,677**	**1.47**	**11,723**	**60,540**	**1.72**	**12,060**	**69,528**	**1.98**	**12,442**
Delaware	840	.44	10,500	1,250	.66	10,684	1,900	11,585	2,288	1.20	10,947	2,675	1.41	10,615	3,063	1.61	10,348	3,450	1.82	10,147
District of Columbia	1,124	.57	7,114	1,598	.81	8,455	1,975	9,975	2,379	1.20	11,895	2,759	1.40	13,138	3,167	1.60	14,461	3,496	1.77	15,333
Maryland	4,880	.49	9,037	7,031	.70	9,618	10,023	10,186	12,338	1.23	10,880	15,576	1.55	11,546	19,557	1.95	12,323	23,734	2.37	13,120
Virginia	6,757	.44	9,504	10,665	.69	10,838	15,519	11,802	19,459	1.25	12,132	23,467	1.51	12,286	27,121	1.75	12,384	30,748	1.98	12,520
West Virginia	3,591	.63	10,289	4,697	.82	9,745	5,717	10,030	6,620	1.16	10,392	7,200	1.26	10,480	7,632	1.33	10,484	8,100	1.42	10,728
South Atlantic (South)	**26,814**	**.49**	**10,337**	**38,833**	**.72**	**10,113**	**54,248**	**10,081**	**67,803**	**1.25**	**10,070**	**82,063**	**1.51**	**10,163**	**95,853**	**1.77**	**10,212**	**108,521**	**2.00**	**10,190**
Florida	7,490	.47	10,685	10,882	.68	9,929	16,054	9,004	22,194	1.38	9,217	28,333	1.76	9,342	34,473	2.15	9,424	39,988	2.49	9,336
Georgia	7,446	.50	11,316	10,680	.72	11,022	14,898	11,703	17,600	1.18	11,146	20,900	1.40	11,188	24,300	1.63	11,266	27,600	1.85	11,316
North Carolina	7,475	.50	9,559	10,890	.72	9,646	15,070	9,941	17,899	1.19	10,118	20,825	1.38	10,188	23,520	1.56	10,199	25,942	1.72	10,201
South Carolina	4,433	.54	9,721	6,381	.78	9,878	8,226	10,168	10,110	1.23	10,348	12,005	1.46	10,624	13,560	1.65	10,719	14,991	1.82	10,824
East North Central	**76,879**	**.59**	**9,261**	**104,338**	**.80**	**9,441**	**130,170**	**9,609**	**154,175**	**1.18**	**9,895**	**180,141**	**1.38**	**10,135**	**208,958**	**1.61**	**10,447**	**231,025**	**1.77**	**10,390**
Illinois	20,312	.61	9,933	29,013	.88	10,403	33,039	9,695	39,600	1.20	10,000	45,000	1.36	10,000	50,500	1.53	10,000	56,000	1.69	10,034
Indiana	9,934	.53	8,661	13,833	.74	9,271	18,597	10,058	22,432	1.21	10,318	26,267	1.41	10,511	30,102	1.62	10,659	33,937	1.82	10,777
Michigan	17,612	.59	9,650	23,483	.78	9,191	30,044	9,574	35,500	1.18	10,323	43,500	1.45	11,337	54,000	1.80	12,685	57,750	1.92	12,272
Ohio	20,027	.58	8,730	26,830	.78	9,046	34,579	9,331	40,000	1.16	9,283	46,000	1.33	9,226	52,250	1.51	9,227	58,500	1.69	9,227
Wisconsin	8,994	.65	9,085	11,179	.80	8,972	13,911	9,620	16,643	1.20	9,796	19,374	1.39	9,920	22,106	1.59	10,016	24,838	1.79	10,097
East South Central	**20,450**	**.55**	**10,174**	**28,548**	**.77**	**9,658**	**36,979**	**9,405**	**45,078**	**1.22**	**9,436**	**52,494**	**1.42**	**9,520**	**59,275**	**1.60**	**9,634**	**65,306**	**1.77**	**9,800**
Alabama	4,890	.49	9,939	7,101	.71	9,727	10,008	9,232	12,048	1.20	9,176	13,836	1.38	9,121	15,804	1.58	9,093	17,568	1.76	9,060
Kentucky	5,620	.55	10,181	7,590	.75	9,256	10,130	9,548	12,650	1.25	9,412	15,200	1.50	9,418	17,060	1.68	9,348	18,550	1.83	9,345
Mississippi	3,847	.65	10,716	5,319	.90	10,429	5,935	8,965	7,250	1.22	9,355	8,475	1.43	9,855	9,575	1.61	10,186	10,500	1.77	10,396
Tennessee	6,093	.56	10,038	8,538	.78	9,529	10,906	9,694	13,130	1.20	9,762	14,983	1.37	9,838	16,836	1.54	10,204	18,688	1.71	10,802
West North Central	**38,684**	**.65**	**8,901**	**50,432**	**.85**	**8,876**	**59,197**	**9,085**	**69,961**	**1.18**	**79,627**	**79,827**	**1.35**	**9,963**	**89,543**	**1.51**	**10,280**	**97,479**	**1.65**	**10,386**
Iowa	7,574	.73	9,192	9,696	.93	8,815	10,382	8,644	12,400	1.19	9,635	14,100	1.36	10,337	16,100	1.55	11,306	17,500	1.69	11,897
Kansas	6,300	.64	8,962	8,167	.83	9,146	9,822	9,214	11,460	1.17	9,582	13,060	1.33	9,842	14,660	1.49	10,027	16,260	1.66	10,182
Minnesota	7,641	.62	8,713	9,909	.80	8,327	12,379	8,773	14,588	1.18	8,944	16,796	1.36	9,079	19,005	1.54	9,298	21,161	1.71	9,481
Missouri	9,737	.64	9,372	12,728	.83	9,679	15,290	9,903	18,329	1.20	10,675	20,901	1.37	10,943	22,951	1.50	10,992	23,890	1.56	10,622
Nebraska	3,932	.66	8,348	5,150	.86	8,456	6,000	9,105	6,960	1.16	9,547	7,925	1.32	10,096	8,890	1.48	10,558	9,790	1.63	10,818
North Dakota	1,576	.67	7,399	2,185	.92	7,776	2,363	7,697	2,775	1.17	8,067	3,180	1.35	8,391	3,592	1.52	8,676	3,960	1.68	8,820
South Dakota	1,924	.65	8,785	2,597	.88	8,833	2,961	9,027	3,449	1.16	9,501	3,865	1.31	9,736	4,345	1.47	9,966	4,918	1.66	10,182
West South Central	**34,254**	**.55**	**9,776**	**48,012**	**.78**	**9,128**	**61,762**	**9,334**	**75,738**	**1.23**	**9,764**	**91,577**	**1.48**	**10,273**	**105,612**	**1.71**	**10,656**	**116,610**	**1.89**	**10,680**
Arkansas	3,375	.53	9,427	4,865	.76	9,730	6,388	10,507	7,590	1.19	11,448	9,102	1.42	11,976	10,615	1.66	12,577	12,127	1.90	11,616
Louisiana	4,709	.53	9,893	6,452	.72	8,707	8,953	8,900	11,046	1.23	8,844	12,926	1.44	8,658	14,782	1.65	8,515	16,644	1.86	8,406
Oklahoma	5,543	.56	8,926	7,552	.77	8,721	9,855	9,341	11,699	1.19	9,669	14,229	1.44	9,985	16,265	1.65	10,307	17,993	1.83	10,628
Texas	20,627	.56	10,067	29,143	.80	9,243	36,566	9,262	45,403	1.24	9,796	55,320	1.51	10,565	63,950	1.75	11,116	69,846	1.91	11,262
Mountain	**15,634**	**.56**	**10,368**	**21,452**	**.77**	**9,822**	**27,808**	**9,563**	**35,244**	**1.27**	**9,820**	**43,318**	**1.56**	**10,086**	**51,937**	**1.87**	**10,338**	**60,667**	**2.18**	**10,447**
Arizona	2,359	.54	12,351	3,380	.77	11,536	4,367	10,109	5,688	1.30	10,085	7,009	1.60	10,056	8,330	1.91	10,048	9,651	2.21	10,032
Colorado	3,813	.57	8,930	5,187	.77	8,762	6,733	8,744	8,426	1.25	9,553	10,168	1.51	10,107	11,910	1.77	10,312	13,652	2.03	10,265
Idaho	1,969	.66	10,309	2,429	.82	8,833	2,977	8,629	3,371	1.13	8,084	3,814	1.28	8,029	4,315	1.45	8,579	4,789	1.61	8,901
Montana	1,560	.59	7,879	2,156	.81	7,783	2,658	7,660	3,140	1.18	8,135	3,624	1.36	8,389	4,108	1.55	8,576	4,589	1.73	8,741
Nevada	722	.50	12,667	1,059	.74	12,607	1,435	11,211	2,069	1.44	12,389	2,759	1.92	13,459	3,476	2.42	14,246	4,135	2.88	14,611
New Mexico	2,008	.49	12,709	2,988	.73	11,764	4,075	11,710	5,203	1.28	10,728	6,659	1.63	10,654	8,546	2.10	10,859	10,800	2.65	11,066
Utah	2,057	.59	11,179	2,695	.77	10,365	3,506	9,685	4,807	1.37	10,163	6,246	1.78	10,290	7,742	2.21	10,392	9,191	2.62	10,468
Wyoming	1,146	.56	11,235	1,558	.76	10,456	2,057	11,688	2,540	1.23	11,814	3,039	1.48	12,254	3,510	1.71	12,447	3,860	1.88	12,254
Pacific	**48,468**	**.61**	**10,266**	**60,426**	**.76**	**9,295**	**79,622**	**9,435**	**94,442**	**1.19**	**9,141**	**121,157**	**1.52**	**9,680**	**154,958**	**1.95**	**10,261**	**198,045**	**2.49**	**10,887**
California	36,100	.59	10,383	45,200	.73	9,297	61,600	9,547	72,300	1.17	9,118	94,800	1.54	9,765	124,500	2.02	10,475	163,300	2.65	11,223
Oregon	5,192	.70	9,759	6,348	.86	9,187	7,370	9,121	9,600	1.30	9,707	11,650	1.58	9,974	13,550	1.84	10,119	15,400	2.09	10,246
Washington	7,176	.67	10,079	8,878	.83	9,365	10,652	9,035	12,542	1.18	8,870	14,707	1.38	8,968	16,908	1.59	9,013	19,345	1.82	9,048
Hawaii	1,026	.64	10,426	1,365	.85	8,922	1,608	8,553	1,870	1.16	8,578	2,129	1.32	8,945	2,388	1.49	9,185	2,646	1.65	9,552
Puerto Rico	762	.53	18,910	1,047	.72	15,997	1,450	11,789	1,963	1.35	11,217	2,436	1.68	12,429	2,843	1.96	13,603	3,157	2.18	14,350
Grand total	**362,477**	**.58**	**9,579**	**486,994**	**.78**	**9,390**	**625,990**	**9,567**	**757,597**	**1.21**	**9,789**	**903,256**	**1.44**	**10,082**	**1,056,643**	**1.69**	**10,389**	**1,206,066**	**1.93**	**10,567**

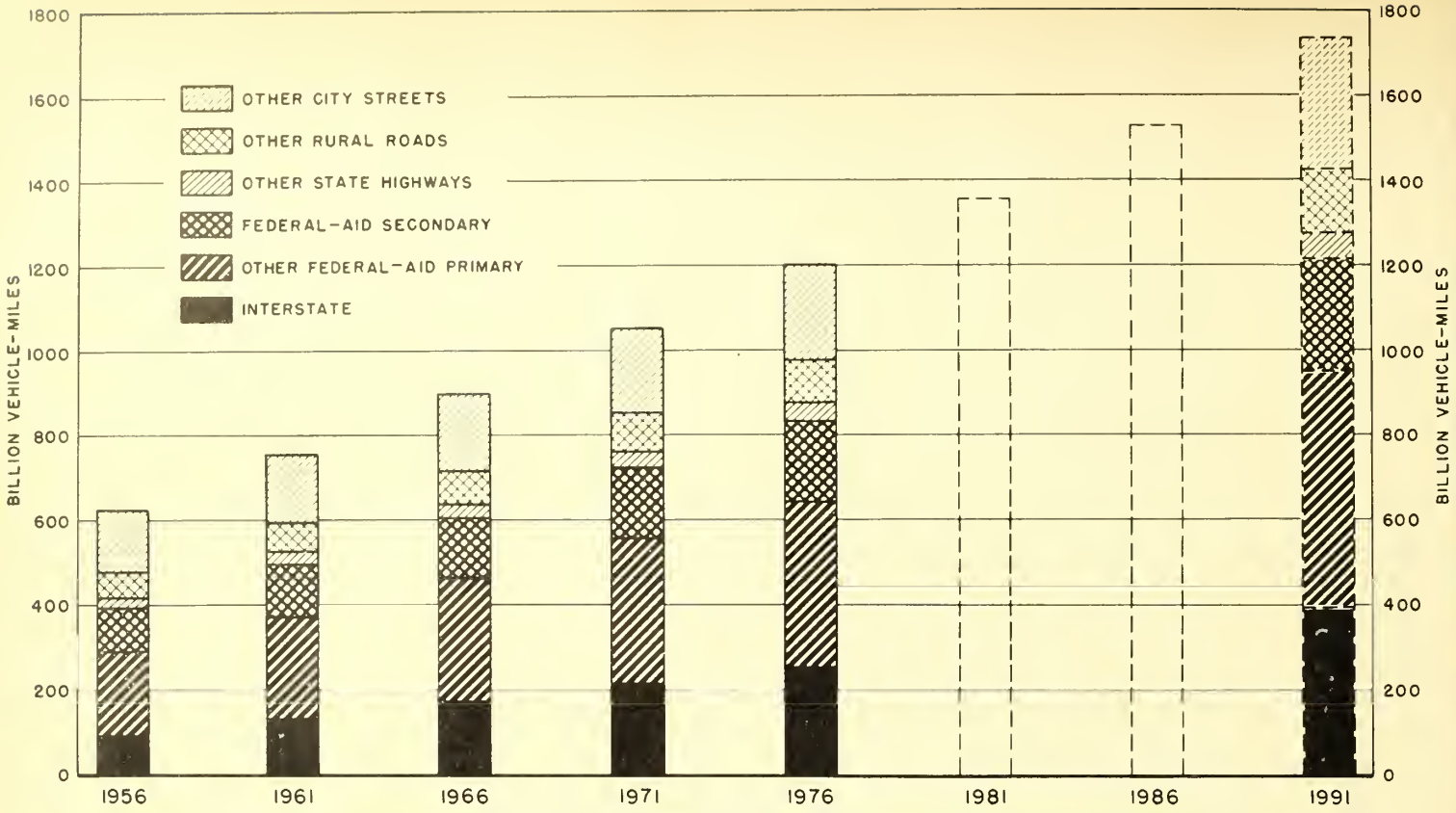

Figure 5.—State estimates of total travel in the United States by highway classifications for selected years, 1956–91.

10 States indicate less annual average travel per motor vehicle in 1976 than in 1956; the decreases ranging from 51 to 1,438 miles per registered vehicle. On the other hand, the travel forecasts for 38 States indicate increases over the 20-year period of up to 3,600 miles per registered motor vehicle.

Travel by road systems

The tabulations of future travel by highway classifications, as prepared by the States, show that a definite shift is expected to occur in the percentage of total travel on the various classes of highways during the forecast period. This information is presented in figure 5 and tables 9 and 10.

In 1956, the Interstate and other Federal-aid primary systems carried 46.6 percent of the Nation's total highway travel. By 1976, travel on these two systems is expected to amount to approximately 54 percent of the total. By far the largest percentage increase in travel will occur on the Interstate System, a growth of from 15 percent in 1956 to 21 percent in 1976.

The percentages of total travel occurring on the Federal-aid secondary system and on State highways not a part of any Federal-aid system are expected to decrease slightly during the forecast period. Much more material decreases are expected to occur on local rural roads and city streets, however. In 1956, travel on local rural roads amounted to 9.9 percent of total travel, and travel on city streets amounted to 23.0 percent of the total. By 1976, these percentages are expected to be 8.6 and 18.4, respectively.

It is not anticipated, however, that there will be any decrease in total vehicle-miles of travel on any class of highways during the 20-year period. In fact, the forecast increases in terms of vehicle-miles are indeed impressive, as may be seen in figure 5 and table 9.

Travel on the Interstate System is expected to almost triple the 1956 figure by 1976, and to be more than four times as great by 1991 (table 9). This increase is the highest anticipated for any of the several highway classifications during the forecast period. On the Federal-aid primary system, excluding the Interstate System, the 1956 travel estimate is expected to almost be doubled by 1976.

Estimates of total travel on all roads and streets show that the States anticipate an increase of 93 percent during the 20-year period, 1956–76. By 1991, total travel is expected to be 2.8 times that reported in 1956.

Among the census divisions (table 10), travel on the Interstate System will constitute a larger percentage of total travel in 1976 than in 1956, varying from 34.2 percent in

Table 9.—State estimates of motor-vehicle travel in the United States (excludes Alaska and Hawaii) by highway classifications for years, 1956, 1976, and 1991

Highway classification	1956		1976			1991		
	Million vehicle-miles	Percent of total	Million vehicle-miles	Percent of total	Ratio: 1976/1956	Million vehicle-miles	Percent of total	Ratio: 1991/1956
Interstate:								
Rural	58,685	9.4	163,640	13.6	2.79	243,315	14.0	4.15
Urban	32,973	5.3	92,244	7.7	2.80	150,450	8.7	4.56
Total	91,658	14.7	255,884	21.3	2.79	393,765	22.7	4.30
Federal-aid Primary:								
Rural	142,510	22.9	267,341	22.3	1.88	370,434	21.4	2.60
Urban	56,148	9.0	120,669	10.0	2.15	186,535	10.7	3.32
Total	198,658	31.9	388,010	32.3	1.95	556,969	32.1	2.80
Federal-aid Secondary:								
Rural	86,294	13.8	155,426	12.9	1.80	215,854	12.5	2.50
Urban	16,620	2.7	33,018	2.8	1.99	48,306	2.8	2.91
Total	102,914	16.5	188,444	15.7	1.83	264,160	15.3	2.57
Other State Highways:								
Rural	14,177	2.3	25,140	2.1	1.77	34,982	2.0	2.47
Urban	10,431	1.7	18,655	1.6	1.79	28,240	1.6	2.71
Total	24,608	4.0	43,795	3.7	1.78	63,222	3.6	2.57
Other Roads and Streets:								
Rural roads	61,539	9.9	103,402	8.6	1.68	149,601	8.6	2.43
City streets	143,555	23.0	220,728	18.4	1.54	305,885	17.7	2.13
Total	205,094	32.9	324,130	27.0	1.58	455,486	26.3	2.22
All highways	622,932	100.0	1,200,263	100.0	1.93	1,733,602	100.0	2.78

Table 10.—Percentage distribution of travel estimated by the States (excludes Alaska and Hawaii) by highway classification and census division, 1956 and 1976

Census division	Percentage distribution of travel								
	Federal-aid systems			Not on Federal-aid systems			Total	All roads and streets	
	Interstate	Other Federal-aid primary	Federal-aid secondary	State highways	Local rural roads	City streets		Rural	Urban
New England:									
1956	12.8	30.2	15.2	10.2	10.5	21.1	100.0	49.6	50.4
1976	20.1	27.5	15.2	9.8	9.8	17.6	100.0	50.2	49.8
Middle Atlantic:									
1956	10.1	31.5	15.3	6.1	13.1	23.9	100.0	50.1	49.9
1976	16.7	32.8	14.6	5.7	12.0	18.2	100.0	50.4	49.6
South Atlantic (North):									
1956	14.8	35.4	22.9	3.1	7.0	16.8	100.0	65.2	34.8
1976	24.6	32.9	19.9	2.7	6.0	13.9	100.0	67.2	32.8
South Atlantic (South):									
1956	12.9	35.3	22.7	4.2	7.4	17.5	100.0	68.6	31.4
1976	21.8	33.9	20.5	3.5	5.7	14.6	100.0	69.8	30.2
East North Central:									
1956	14.7	28.3	13.4	3.5	9.3	30.8	100.0	52.2	47.8
1976	19.8	29.4	13.3	3.1	7.5	26.9	100.0	55.0	45.0
East South Central:									
1956	17.7	35.4	18.4	2.1	8.8	17.6	100.0	67.7	32.3
1976	23.1	36.0	19.0	.6	6.7	14.6	100.0	68.7	31.3
West North Central:									
1956	11.6	40.2	14.6	.5	9.9	23.2	100.0	66.2	33.8
1976	21.8	37.0	14.7	.4	6.5	19.6	100.0	67.5	32.5
West South Central:									
1956	15.8	34.1	18.8	4.3	7.1	19.9	100.0	64.2	35.8
1976	24.4	33.3	16.5	3.6	5.5	16.7	100.0	64.5	35.5
Mountain:									
1956	27.6	30.9	16.9	1.9	8.4	14.3	100.0	74.5	25.5
1976	34.2	31.8	15.6	2.5	5.1	10.8	100.0	77.1	22.9
Pacific:									
1956	18.5	26.0	15.4	3.2	12.4	24.5	100.0	52.2	47.8
1976	20.0	31.7	14.6	3.8	13.4	16.5	100.0	53.2	46.8
All census divisions:									
1956	14.7	31.9	16.5	4.0	9.9	23.0	100.0	58.3	41.7
1976	21.3	32.3	15.7	3.7	8.6	18.4	100.0	59.6	40.4

the Mountain division to 16.7 in the Middle Atlantic division.

For the forecast period, the West North Central division predicts the largest relative increase of travel on the Interstate System, rising from 11.6 percent of all travel in 1956 to 21.8 percent in 1976. The Pacific division estimates the smallest increase, rising from 18.5 to 20.0 percent, during the forecast period.

Moderate fluctuations appear in the distribution of travel on the Federal-aid primary routes, excluding the Interstate System, over the 20-year period. Half of the census divisions estimate a percentage decrease from 1956 to 1976 in the amount of travel, while the other half predict a percentage increase. The Pacific division is expected to have an increase of 5.7 percentage points, whereas a decrease of 3.2 percentage points is predicted for the West North Central division. It will be noted that these two divisions occupied practically opposite positions in the 1976 travel estimates for the Interstate System. For all census divisions a slight percentage increase is anticipated for travel on the other Federal-aid primary routes.

The proportion of total travel on the Federal-aid secondary system is expected to decrease in seven of the census divisions, increase in two divisions, and remain the same in the New England division. For all census divisions, 15.7 percent of all travel for 1976 will be on the Federal-aid secondary system. In percentage points, this represents a 0.8 decrease from the 1956 travel estimate.

In the Mountain and Pacific divisions it is expected that State highways not on the Federal-aid system will carry a larger percentage of total travel in 1976 than they did in 1956, while in the other eight divisions percentage decreases are expected. This class of highways is expected to carry as much as 9.8 percent of the 1976 total travel in the New England division and as little as 0.4 percent in the West North Central division. The extent to which Federal-aid and State highway mileages coincide in an individual State is, of course, an important factor in determining how much travel will be performed on State highways not a part of any Federal-aid system.

Of the 10 census divisions, only in the Pacific division is it anticipated that local rural roads not on the Federal-aid systems will carry a larger percentage of total travel in 1976 than in 1956—a rise from 12.4 to 13.4 percent. This latter percentage figure for local road travel in relation to total travel is predicted to be the greatest among the census divisions in 1976. Estimates for the other nine divisions show percentage point decreases in local road travel ranging from 3.4 in the West North Central to 0.7 in New England. The smallest percentage of travel in 1976 on local rural roads, 5.1 percent, is anticipated in the Mountain division.

In 1976, as compared to 1956, the percentage of total travel on city streets which are not a part of the Federal-aid systems is expected to decrease in all census divisions. The decreases range from 8.0 percentage points in the Pacific division to 2.9 in both the South Atlantic (North) and South Atlantic (South) divisions. The expected range of travel on city streets is from 26.9 percent of total travel in the East North Central division to only 10.8 percent of the travel in the Mountain division.

Of the total 1956 travel on all classes of highways 58.3 percent took place on the rural roads, and 41.7 percent of the travel was carried on urban roads and streets. The forecasts of 1976 travel show that there will be little change in these percentage distributions of travel. However, it appears that there will be a substantial shift of travel from city streets not on any Federal-aid system to those which are a part of the Federal-aid systems.

Motor-Fuel Consumption

According to the State estimates, as summarized by census divisions in table 11 and figure 6, consumption of motor fuel is expected to increase 94 percent during the 1956–76 period, an increase of 47 billion gallons.

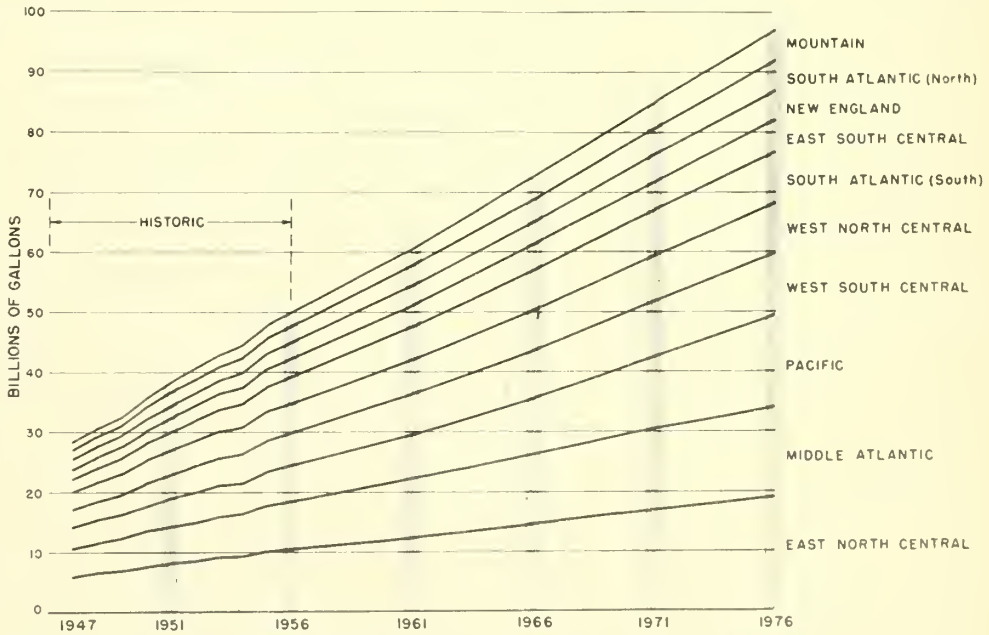

Figure 6.—State estimates of motor-fuel consumption in the United States by census divisions for selected years, 1947–76.

Table 11.—State estimates of motor-fuel consumption in the United States (excludes Alaska and Hawaii) by census division, 1956 and 1976

Census division	1956				1976					
	Total gallons	Percent of total	Gallons per registered vehicle	Miles per gallon	Total gallons	Percent of total	Ratio: 1976/1956	Gallons per registered vehicle	Ratio: 1976/1956	Miles per gallon
	Millions				*Millions*					
New England	2,730	5.5	759	12.59	4,874	5.0	1.79	802	1.06	12.63
Middle Atlantic	7,997	16.0	730	12.96	14,916	15.4	1.87	823	1.13	12.84
South Atlantic (North)	2,713	5.4	840	12.95	5,111	5.3	1.88	915	1.09	13.60
South Atlantic (South)	4,347	8.7	808	12.48	8,703	9.0	2.00	817	1.01	12.47
East North Central	10,356	20.7	764	12.57	19,078	19.6	1.84	858	1.12	12.11
East South Central	3,109	6.2	791	11.89	5,189	5.3	1.67	779	.98	12.59
West North Central	4,894	9.8	751	12.10	8,299	8.5	1.70	884	1.18	11.75
West South Central	5,340	10.7	807	11.57	10,314	10.6	1.93	945	1.17	11.31
Mountain	2,345	4.7	806	11.86	5,210	5.4	2.22	897	1.11	11.64
Pacific	6,180	12.3	732	12.88	15,450	15.9	2.50	849	1.16	12.82
All census divisions	50,011	100.0	768	12.46	97,144	100.0	1.94	855	1.11	12.36

The magnitudes of such figures are difficult to comprehend. Visualize a lake 1 mile square filled with motor fuel to a depth of about 240 feet, and this would be the gallons of motor fuel consumed in 1956; fill the same lake to a depth of about 465 feet and you would have the gallonage which is expected to be consumed in 1976.

The estimated percentage increase in motor-fuel consumption is but one percentage point greater than the anticipated increase in total travel. The closeness of the two forecasts indicates that there is expected to be very little change in the overall miles-per-gallon value during the forecast period. In 1956, the estimated average miles-per-gallon value was 12.46; in 1976, the average value is expected to be 12.36 miles per gallon. There is

Table 12.—State forecasts of motor-fuel consumption in the United States by census division and State for selected years, 1947–76

Census division and State	1947				1951				1956			1961			
	Total gallons	Ratio: 1947/1956	Gallons per registered vehicle	Miles per gallon	Total gallons	Ratio: 1951/1956	Gallons per registered vehicle	Miles per gallon	Total gallons	Gallons per registered vehicle	Miles per gallon	Total gallons	Ratio: 1961/1956	Gallons per registered vehicle	Miles per gallon
	Millions				*Millions*				*Millions*			*Millions*			
United States	**28,244**	**0.56**	**749**	**12.8**	**38,207**	**0.76**	**740**	**12.7**	**50,011**	**768**	**12.4**	**60,690**	**1.21**	**788**	**12.4**
New England	**1,694**	**.62**	**720**	**12.8**	**2,131**	**.78**	**722**	**12.7**	**2,730**	**759**	**12.6**	**3,262**	**1.19**	**774**	**12.7**
Connecticut	405	.59	630	13.0	523	.76	684	13.0	685	717	13.0	852	1.24	738	13.0
Maine	190	.67	792	13.1	228	.81	826	13.1	283	832	13.1	309	1.09	824	13.2
Massachusetts	769	.62	734	12.6	979	.79	727	12.2	1,234	762	12.1	1,480	1.20	776	12.2
New Hampshire	108	.61	706	13.1	134	.76	744	14.5	176	782	14.3	220	1.25	827	14.1
Rhode Island	141	.61	675	13.0	167	.72	640	13.0	231	726	12.6	262	1.13	738	12.9
Vermont	81	.67	750	12.7	100	.83	813	12.4	121	858	11.8	139	1.15	880	11.8
Middle Atlantic	**4,583**	**.57**	**702**	**13.2**	**6,126**	**.77**	**703**	**13.1**	**7,997**	**730**	**13.0**	**9,752**	**1.22**	**752**	**13.0**
New Jersey	947	.53	756	13.4	1,306	.73	769	13.3	1,792	796	13.4	2,272	1.27	851	13.4
New York	1,943	.59	674	13.3	2,560	.77	668	13.1	3,313	689	12.7	4,070	1.23	711	12.8
Pennsylvania	1,693	.59	707	13.0	2,260	.78	709	13.0	2,892	743	13.0	3,410	1.18	746	13.0
South Atlantic (North)	**1,440**	**.53**	**783**	**11.9**	**2,047**	**.75**	**818**	**12.3**	**2,713**	**840**	**13.0**	**3,269**	**1.20**	**864**	**13.2**
Delaware	68	.48	850	12.4	98	.70	838	12.8	141	860	13.5	181	1.28	866	12.6
District of Columbia	153	.78	968	7.3	201	1.02	1,063	8.0	197	995	10.0	203	1.03	1,015	11.7
Maryland	375	.49	694	13.0	541	.70	740	13.0	771	784	13.0	940	1.22	829	13.1
Virginia	567	.50	797	11.9	830	.73	843	12.8	1,140	867	13.6	1,418	1.24	884	13.7
West Virginia	277	.60	794	13.0	377	.81	782	12.5	464	814	12.3	527	1.14	827	12.6
South Atlantic (South)	**2,149**	**.49**	**827**	**12.5**	**3,130**	**.72**	**815**	**12.4**	**4,347**	**808**	**12.5**	**5,449**	**1.25**	**809**	**12.4**
Florida	591	.47	843	12.7	868	.69	792	12.5	1,265	709	12.7	1,749	1.38	726	12.7
Georgia	540	.48	821	13.8	778	.69	803	13.7	1,124	883	13.3	1,355	1.21	858	13.0
North Carolina	682	.52	872	11.0	997	.76	883	10.9	1,309	863	11.5	1,550	1.18	876	11.5
South Carolina	336	.52	737	13.2	487	.75	754	13.1	649	802	12.7	795	1.22	814	12.7
East North Central	**6,034**	**.58**	**727**	**12.7**	**8,088**	**.78**	**732**	**12.9**	**10,356**	**764**	**12.6**	**12,436**	**1.20**	**798**	**12.4**
Illinois	1,569	.60	767	12.9	2,161	.83	775	13.4	2,606	765	12.7	3,125	1.20	789	12.7
Indiana	837	.53	730	11.9	1,165	.74	781	11.9	1,581	855	11.8	1,869	1.18	860	12.0
Michigan	1,345	.59	737	13.1	1,762	.77	690	13.3	2,278	726	13.2	2,660	1.17	773	13.3
Ohio	1,589	.56	693	12.6	2,128	.75	717	12.6	2,832	764	12.2	3,494	1.23	811	11.4
Wisconsin	694	.66	701	13.0	872	.82	700	12.8	1,059	732	13.1	1,288	1.22	758	12.9
East South Central	**1,629**	**.52**	**810**	**12.6**	**2,262**	**.73**	**765**	**12.6**	**3,109**	**791**	**11.9**	**3,687**	**1.19**	**772**	**12.2**
Alabama	404	.48	821	12.1	582	.70	797	12.2	834	769	12.0	1,004	1.20	765	12.0
Kentucky	431	.56	781	13.0	572	.74	698	13.3	771	727	13.1	960	1.25	714	13.2
Mississippi	317	.56	883	12.1	442	.78	867	12.0	565	853	10.5	688	1.22	888	10.5
Tennessee	477	.51	786	12.8	666	.71	743	12.8	939	835	11.6	1,035	1.10	770	12.7
West North Central	**3,011**	**.62**	**694**	**12.8**	**4,001**	**.82**	**704**	**12.6**	**4,894**	**751**	**12.1**	**5,849**	**1.20**	**805**	**12.0**
Iowa	567	.63	688	13.4	750	.83	682	12.9	899	749	11.5	1,116	1.24	867	11.1
Kansas	455	.61	647	13.8	590	.79	661	13.8	744	698	13.2	910	1.22	761	12.6
Minnesota	583	.60	665	13.1	733	.75	616	13.5	977	692	12.7	1,153	1.18	707	12.7
Missouri	803	.58	773	12.1	1,127	.82	857	11.3	1,380	894	11.1	1,620	1.17	944	11.3
Nebraska	331	.67	703	11.9	429	.87	704	12.0	492	747	12.2	585	1.19	802	11.9
North Dakota	126	.66	592	12.5	176	.92	626	12.4	191	622	12.4	226	1.18	657	12.3
South Dakota	149	.71	680	12.9	196	.93	667	13.3	211	643	14.0	239	1.13	658	14.4
West South Central	**2,802**	**.52**	**800**	**12.2**	**4,017**	**.75**	**764**	**12.0**	**5,340**	**807**	**11.6**	**6,573**	**1.23**	**847**	**11.5**
Arkansas	277	.56	774	12.2	383	.77	766	12.7	498	819	12.8	598	1.20	902	12.7
Louisiana	381	.48	800	12.4	537	.68	725	12.0	788	783	11.4	948	1.20	759	11.7
Oklahoma	439	.56	707	12.6	605	.77	699	12.5	789	748	12.5	937	1.19	774	12.5
Texas	1,705	.52	832	12.1	2,492	.76	790	11.7	3,265	827	11.2	4,090	1.25	882	11.1
Mountain	**1,254**	**.53**	**832**	**12.5**	**1,726**	**.74**	**790**	**12.4**	**2,345**	**806**	**11.9**	**3,010**	**1.28**	**839**	**11.7**
Arizona	177	.48	927	13.3	252	.69	860	13.4	367	850	11.9	478	1.30	848	11.9
Colorado	293	.56	686	13.0	393	.75	664	13.2	524	681	12.8	665	1.27	754	12.7
Idaho	169	.63	885	11.7	214	.79	778	11.4	270	783	11.0	327	1.21	784	10.3
Montana	141	.59	712	11.1	195	.81	704	11.1	240	692	11.1	284	1.18	736	11.1
Nevada	66	.44	1,158	10.9	94	.63	1,119	11.3	150	1,172	9.6	216	1.44	1,293	9.6
New Mexico	155	.46	981	13.0	232	.69	913	12.9	335	963	12.2	444	1.33	915	11.7
Utah	162	.55	880	12.7	216	.74	831	12.5	293	809	12.0	394	1.34	833	12.2
Wyoming	91	.55	892	12.6	130	.78	872	12.0	166	943	12.4	202	1.22	940	12.6
Pacific	**3,645**	**.59**	**772**	**13.3**	**4,679**	**.76**	**720**	**12.9**	**6,180**	**732**	**12.9**	**7,403**	**1.20**	**717**	**12.8**
California	2,735	.58	787	13.2	3,505	.74	721	12.9	4,741	735	13.0	5,564	1.17	702	13.0
Oregon	387	.65	727	13.4	504	.84	729	12.6	598	740	12.3	768	1.28	777	12.5
Washington	523	.62	735	13.7	670	.80	707	13.3	841	713	12.7	1,071	1.27	757	11.7
Hawaii	73	.63	745	14.0	97	.84	634	14.1	115	612	14.0	134	1.17	615	14.0
Puerto Rico	64	.52	1,600	12.0	87	.71	1,338	12.0	123	1,000	11.8	164	1.33	937	12.0
Grand total	28,381	.56	750	12.8	38,391	.76	740	12.7	50,249	768	12.5	60,988	1.21	788	12.4

speculation, of course, on what effect the growing number of compact cars will have on motor-fuel consumption, but it is believed by some that any effect will be slight and will cause only minor variations in the average miles-per-gallon value.

Forecasts by divisions and States

A comparison of the State estimates of motor-fuel consumption by census divisions (table 11) shows that consumption in States of the Pacific division is expected to be 2½ times as much in 1976 as in 1956. The Mountain and South Atlantic (South) divisions anticipate at least doubling their motor-fuel consumption during the 20-year period. These estimated increases generally parallel the growth in the population and economy of these areas. The lowest rate of increase, 67 percent, is expected to occur in the East South Central division, followed in order by the West North Central (70 percent) and the New England (79 percent) divisions. Again, these anticipated increases are generally in line, possibly not with the future industrial growth of these areas, but certainly with the forecasts of population.

A review of the motor-fuel consumption forecasts prepared by individual States (table 12) shows a wide divergence in the predicted percentage increases, ranging from a high of 175 percent predicted by New Mexico and Nevada to a low of 39 percent predicted by West Virginia. The lowest predicted increase, however, was submitted by the District of Columbia, which expects only a 17-percent rise in fuel consumption. The estimated percentage increases in total travel for the two highest States, Nevada, 188 percent and New Mexico, 165 percent, are reasonably well-alined with the fuel consumption forecasts. California, Utah, and Florida are the next highest in anticipated percentage increases in motor-fuel consumption, the percentage increases agreeing exactly with their travel projections.

Fuel consumption per vehicle

Estimates of motor-fuel consumption per registered vehicle are shown in table 11 by census division and in table 12 for the individual

Table 12.—State forecasts of motor-fuel consumption in the United States by census division and State for selected years, 1917–76—(*Continued*)

Census division and State	1966				1971				1976				
	Total gallons	Ratio: 1966/1956	Gallons per registered vehicle	Miles per gallon	Total gallons	Ratio: 1971/1956	Gallons per registered vehicle	Miles per gallon	Total gallons	Ratio: 1976/1956	Gallons per registered vehicle	Ratio: 1976/1956	Miles per gallon
	Millions				*Millions*				*Millions*				
United States	72,605	1.45	814	12.4	85,073	1.70	840	12.4	97,144	1.94	855	1.11	12.4
New England	3,811	1.40	786	12.7	4,351	1.59	796	12.7	4,874	1.79	802	1.06	12.6
Connecticut	1,015	1.48	749	13.0	1,178	1.72	758	13.0	1,341	1.96	764	1.07	13.0
Maine	338	1.19	824	13.1	365	1.29	824	13.2	395	1.40	830	1.00	13.1
Massachusetts	1,750	1.42	792	12.2	2,020	1.64	805	12.2	2,275	1.84	810	1.06	12.2
New Hampshire	260	1.48	861	13.9	294	1.67	886	13.9	325	1.85	903	1.15	13.9
Rhode Island	233	1.27	740	13.0	324	1.40	741	12.8	355	1.54	744	1.02	12.6
Vermont	155	1.28	891	11.8	170	1.40	904	11.8	183	1.51	924	1.08	11.7
Middle Atlantic	11,420	1.43	771	13.0	13,187	1.65	801	12.9	14,916	1.87	823	1.13	12.8
New Jersey	2,675	1.49	886	13.4	3,122	1.74	924	13.4	3,606	2.01	962	1.21	13.4
New York	4,875	1.47	753	12.7	5,795	1.75	808	12.5	6,700	2.02	839	1.22	12.4
Pennsylvania	3,870	1.34	729	13.0	4,270	1.48	721	13.0	4,610	1.59	721	.97	13.0
South Atlantic (North)	3,878	1.43	880	13.3	4,498	1.66	896	13.5	5,111	1.88	915	1.09	13.6
Delaware	215	1.52	853	12.4	249	1.77	841	12.3	284	2.01	835	.97	12.1
District of Columbia	213	1.08	1,014	13.0	222	1.13	1,014	14.3	231	1.17	1,013	1.02	15.1
Maryland	1,175	1.52	871	13.3	1,461	1.89	921	13.4	1,756	2.28	971	1.24	13.5
Virginia	1,698	1.49	889	13.8	1,950	1.71	890	13.9	2,196	1.93	894	1.03	14.0
West Virginia	577	1.24	840	12.5	616	1.33	846	12.4	644	1.39	853	1.05	12.6
South Atlantic (South)	6,595	1.52	817	12.1	7,691	1.77	819	12.5	8,703	2.00	817	1.01	12.5
Florida	2,233	1.77	736	12.7	2,717	2.15	743	12.7	3,151	2.49	736	1.04	12.7
Georgia	1,611	1.43	862	13.0	1,868	1.66	866	13.0	2,123	1.89	870	.99	13.0
North Carolina	1,803	1.38	882	11.6	2,036	1.56	883	11.6	2,246	1.72	883	1.02	11.6
South Carolina	948	1.46	839	12.7	1,070	1.65	846	12.7	1,183	1.82	854	1.06	12.7
East North Central	14,690	1.42	826	12.3	17,138	1.65	857	12.2	19,078	1.84	858	1.12	12.1
Illinois	3,552	1.36	789	12.7	3,986	1.53	789	12.7	4,420	1.70	792	1.04	12.7
Indiana	2,189	1.38	876	12.0	2,509	1.59	888	12.0	2,828	1.79	898	1.05	12.0
Michigan	3,260	1.43	850	13.3	4,047	1.78	951	13.3	4,327	1.90	919	1.27	13.3
Ohio	4,172	1.47	837	11.0	4,851	1.71	857	10.8	5,529	1.95	872	1.14	10.6
Wisconsin	1,517	1.43	777	12.8	1,745	1.65	791	12.7	1,974	1.86	802	1.10	12.6
East South Central	4,276	1.38	775	12.3	4,787	1.54	778	12.4	5,189	1.67	779	.98	12.6
Alabama	1,153	1.38	760	12.0	1,317	1.58	758	12.0	1,464	1.76	755	.98	12.0
Kentucky	1,141	1.48	707	13.3	1,283	1.66	703	13.3	1,395	1.81	703	.97	13.3
Mississippi	789	1.40	917	10.7	867	1.53	922	11.0	934	1.65	925	1.08	11.2
Tennessee	1,193	1.27	783	12.6	1,320	1.41	800	12.8	1,396	1.49	807	.97	13.4
West North Central	6,734	1.38	840	11.9	7,578	1.55	870	11.8	8,299	1.70	884	1.18	11.7
Iowa	1,298	1.44	952	10.9	1,496	1.66	1,051	10.8	1,624	1.81	1,104	1.47	10.8
Kansas	1,079	1.45	813	12.1	1,222	1.64	836	12.0	1,355	1.82	848	1.21	12.0
Minnesota	1,329	1.36	718	12.6	1,504	1.54	736	12.6	1,676	1.72	751	1.09	12.6
Missouri	1,834	1.33	960	11.4	2,013	1.46	964	11.4	2,154	1.56	958	1.07	11.1
Nebraska	666	1.35	848	11.9	747	1.52	887	11.9	823	1.67	909	1.22	11.9
North Dakota	259	1.36	683	12.3	294	1.54	710	12.2	325	1.70	724	1.16	12.2
South Dakota	269	1.27	678	14.4	302	1.43	693	14.4	342	1.62	708	1.10	14.4
West South Central	7,998	1.50	897	11.4	9,286	1.74	937	11.4	10,314	1.93	945	1.17	11.3
Arkansas	722	1.45	950	12.6	849	1.70	1,006	12.5	978	1.96	937	1.14	12.4
Louisiana	1,108	1.41	742	11.7	1,268	1.61	730	11.7	1,428	1.81	721	.92	11.7
Oklahoma	1,139	1.44	799	12.5	1,302	1.65	825	12.5	1,441	1.83	851	1.14	12.5
Texas	5,029	1.54	960	11.0	5,867	1.80	1,020	10.9	6,467	1.98	1,043	1.26	10.8
Mountain	3,707	1.58	863	11.7	4,444	1.90	885	11.7	5,210	2.22	897	1.11	11.6
Arizona	589	1.60	845	11.9	700	1.91	844	11.9	811	2.21	843	.99	11.9
Colorado	803	1.53	798	12.7	940	1.79	814	12.7	1,078	2.06	811	1.19	12.7
Idaho	384	1.42	808	9.9	442	1.64	879	9.8	499	1.85	928	1.19	9.6
Montana	328	1.37	759	11.0	371	1.55	775	11.1	415	1.73	790	1.04	11.1
Nevada	282	1.88	1,376	9.8	347	2.31	1,422	10.0	413	2.75	1,459	1.24	10.0
New Mexico	568	1.70	909	11.7	728	2.17	925	11.7	921	2.75	944	.98	11.7
Utah	515	1.76	848	12.1	642	2.19	862	12.1	765	2.61	871	1.08	12.0
Wyoming	238	1.43	960	12.8	274	1.65	972	12.8	308	1.86	978	1.04	12.5
Pacific	9,496	1.54	759	12.8	12,113	1.96	802	12.8	15,150	2.50	849	1.16	12.8
California	7,298	1.54	752	13.0	9,574	2.02	805	13.0	12,559	2.65	863	1.17	13.0
Oregon	932	1.56	798	12.5	1,084	1.81	810	12.5	1,226	2.05	816	1.10	12.6
Washington	1,266	1.51	772	11.6	1,455	1.73	776	11.6	1,665	1.98	779	1.09	11.6
Hawaii	152	1.32	639	14.0	171	1.49	658	14.0	189	1.64	682	1.11	14.0
Puerto Rico	203	1.65	1,036	12.0	237	1.93	1,134	12.0	263	2.14	1,195	1.20	12.0
Grand total	72,960	1.45	814	12.4	85,481	1.70	840	12.4	97,596	1.94	855	1.11	12.4

Table 13.—State forecasts of motor-fuel consumption per capita, based on total population and persons 15 to 74 years of age for selected years, 1947–76

Year	Gallons of motor fuel consumed	All ages			Driving age, 15–74		
		Persons	Per capita		Persons	Per capita	
			Gallons of motor fuel consumed	Index, 1956=100		Gallons of motor fuel consumed	Index, 1956=100
	Millions	*Thousands*			*Thousands*		
1947	28,244	144,261	196	65.6	100,542	281	63.7
1951	38,207	153,440	249	83.3	105,974	361	81.9
1956	50,011	167,250	299	100.0	113,420	441	100.0
1961	60,690	180,656	336	112.4	121,041	501	113.6
1966	72,605	195,353	372	124.4	131,360	553	125.4
1971	85,073	211,653	402	134.4	142,858	596	135.1
1976	97,144	229,758	423	141.5	154,320	629	142.6

States. Since these figures were derived by dividing the estimated total motor-fuel consumption by the estimated motor-vehicle registrations, they are subject to the same reservations that were expressed in connection with the figures for average annual travel per registered vehicle. Again, the census division and national figures may be considered as entirely reasonable.

In 1956, the average fuel consumption per registered motor vehicle in the South Atlantic (North) division was 840 gallons, the highest among the 10 census divisions. The lowest figure, 730 gallons, was in the Middle Atlantic division. By 1976, the West South Central division is expected to have the highest consumption rate per registered vehicle, 945 gallons. While this represents a 17-percent increase over 1956, the West North Central division will have a slightly greater increase of 18 percent during the forecast period—highest of all the divisions. In the East South Central division an actual decline in the gallons consumed per vehicle was indicated, from 791 to 779 gallons. The latter was the lowest 1976 value reported for the divisions.

Among the individual States, the 1956 motor-fuel consumption per registered vehicle varied from 1,172 gallons in Nevada to 622 gallons in North Dakota. By 1976 the variations are expected to range from 1,459 gallons, again in Nevada, to 703 in Kentucky.

The changing figures for average motor-fuel consumption reveal an actual decline in fuel consumption per vehicle in 9 States. A decrease of 62 gallons per vehicle during the forecast period is indicated in Louisiana. Eleven States show increases of 50 gallons or less per vehicle; 11 other States, increases of 51 to 100 gallons; and 17 States, increases of over 100 gallons per vehicle. Iowa's forecasts indicate a usage of 355 more gallons of fuel per registered vehicle in 1976 than in 1956.

Total travel in the District of Columbia, recognizedly in an unusual situation since it is a city rather than a State, will increase 77 percent during the 20-year forecast period as compared with a 17-percent increase in fuel consumption. The results of relating these two forecasts are reflected in a 51-percent increase in the miles-per-gallon value, from 10.0 in 1956 to 15.1 in 1976. It seems obvious that this increase is an artificial value. The motor-fuel consumption forecast prepared by the District was based on historic data of motor-fuel taxed and motor-vehicles registered in the District, and the recognition that an increasing proportion of the motor-fuel consumed in traveling on the District's highways is being purchased outside of the District. A somewhat parallel situation exists with regard to the figures for annual travel per registered vehicle in the District of Columbia.

Per capita consumption rate

Probably the most noteworthy increase in motor-fuel consumption is expected to occur in the consumption-per-capita values, shown in table 13. The anticipated increase for the forecast period (1956–76) of 124 gallons per person, or 42 percent, may seem rather optimistic, but on a percentage basis it is less than the 53-percent increase from 1947 to 1956. Similar results are obtained when the per capita consumption rates for the driver age group are compared.

Area distribution expected to shift

As noted in the discussions concerning forecasts of population and registrations, the changing figures for total motor-fuel consumption within each geographical area indicate a definite shift westward during the 20-year period. In 1956, the 26 States (and the District of Columbia) located east of the Mississippi River accounted for 62.5 percent of the total motor-fuel consumption; by 1976, this value is expected to be 59.6 percent.

A review of the motor-fuel estimates by census divisions establishes that there may be a very close relationship between the levels of motor-fuel consumption and population, registrations, and travel, both in 1956 and 1976. Table 14 shows the percentages of national totals for each division for each of the above-mentioned items for the two study years. It is to be expected that these items would be closely related, since population must always be considered as the key factor in future highway use and planning. The movement of people and the movement of the goods and services are the predominant factors of traffic generation.

Interdependence of Basic Forecasting Factors

The interdependence of the various related factors used by the States in making their projections can be partially demonstrated by the distribution of motor-fuel consumption per vehicle, which is derived from a State's estimates of total fuel consumption, registrations, and travel. If a large increase is shown for the fuel consumption per vehicle, then in all probability it will be found that the State has a declining miles-per-gallon rate, a substantial increase in annual travel per vehicle, and only moderate increases in registrations and travel. For example, Iowa's forecast of motor-fuel consumption indicates a usage of 355 more gallons per vehicle in 1976 than in 1956. A review of the Iowa projection shows the miles-per-gallon value decreasing from 11.5 to 10.8, annual travel per vehicle increasing 38 percent, with registrations increasing only 22 percent, and total travel, 69 percent.

Similarly, a decline in a State's motor-fuel consumption per vehicle rate will in all probability show an increase in the miles-per-gallon value, a decreasing rate of annual average travel per vehicle, and very optimistic forecasts of registrations and total travel for that State. A review of the Louisiana projection shows a decline in the gallons of fuel consumed, whereas an 86-percent increase of total travel, a 97-percent increase in registrations, a 6-percent decrease in annual average travel per vehicle, and an increase in mile-per-gallon values from 11.4 in 1956 to 11.7 in 1976 are anticipated. The above observations are rather general, and exceptions to them can be expected.

(Continued on page 282)

Table 14.—Percentage distribution of population, motor-vehicle registrations, travel, and motor-fuel consumption in the United States (excludes Alaska and Hawaii) by census division, 1956 and 1976

Census division	1956				1976			
	Population	Motor-vehicle registrations	Motor vehicle travel	Motor-fuel consumption	Population	Motor-vehicle registrations	Motor-vehicle travel	Motor-fuel consumption
New England	5.9	5.5	5.5	5.5	5.3	5.3	5.1	5.0
Middle Atlantic	19.5	16.8	16.7	16.0	17.9	15.9	16.0	15.4
South Atlantic (North)	5.8	5.0	5.6	5.4	5.5	4.9	5.8	5.3
South Atlantic (South)	8.6	8.3	8.7	8.7	9.3	9.4	9.0	9.0
East North Central	20.4	20.8	20.9	20.7	20.0	19.6	19.3	19.6
East South Central	7.1	6.0	5.9	6.2	5.8	5.9	5.4	5.3
West North Central	9.0	10.0	9.5	9.8	7.7	8.3	8.1	8.5
West South Central	9.6	10.1	9.9	10.7	9.3	9.6	9.7	10.6
Mountain	3.7	4.5	4.5	4.7	4.4	5.1	5.1	5.4
Pacific	10.4	13.0	12.8	12.3	14.8	16.0	16.5	15.9
Total	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0

Estimated Travel by Motor Vehicles in the United States, 1958

BY THE DIVISION OF HIGHWAY PLANNING
BUREAU OF PUBLIC ROADS

Reported by ALEXANDER FRENCH,
Highway Research Engineer

TOTAL motor-vehicle travel in 1958 amounted to 664.7 billion vehicle-miles, an increase of 2.7 percent over the 647.0 billion figure for 1957. For 1959 the total is estimated at 696 billion vehicle-miles, based on reports for the first three quarters of the 1959 calendar year.

Of the 1958 travel, 40 percent was on main rural roads, which constitute 14 percent of the Nation's 3.5 million miles of roads and streets. Another 14 percent of the travel was on local rural roads, which comprise 75 percent of all mileage. The remaining 46 percent of travel was on urban streets, which include only 11 percent of the total mileage.

The average motor vehicle traveled 9,658 miles in 1958, almost half of it in cities, and averaged 12.44 miles per gallon of fuel. Compared to 1957,[1] it appears that the average motor vehicle traveled 87 miles further with no significant change in the miles per gallon of fuel consumed.

In 1958, passenger cars represented 83 percent of the vehicles and performed 82 percent of the travel; the same percentages as reported for the preceding year. The average passenger car in 1958 traveled 9,494 miles, an increase of 1.1 percent over the 9,391-mile average in 1957; and consumed 664 gallons of fuel at a rate of 14.30 miles per gallon, indicating a slight increase in the rate of fuel consumption compared to the previous year.

[1] See previous articles on motor-vehicle travel data in PUBLIC ROADS; the most recent article, for 1957, appears in vol. 30, No. 10, October 1959.

Table 1.—Estimate of motor-vehicle travel in the United States, by vehicle types, in the calendar year 1958

Vehicle type	Motor-vehicle travel					Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption		Average travel per gallon of fuel consumed
	Main rural road travel	Local rural road travel	Total rural travel	Urban travel	Total travel			Total	Average per vehicle	
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousands*	*Miles*	*Million gallons*	*Gallons*	*Miles/gal.*
Passenger cars [1]	208,365	72,888	281,253	263,620	544,873	57,392	9,494	38,095	664	14.30
Buses:										
Commercial	910	150	1,060	1,854	2,914	84	34,690	618	7,357	4.72
School and nonrevenue	567	574	1,141	255	1,396	186	7,505	191	1,027	7.31
All buses [2]	1,477	724	2,201	2,109	4,310	270	15,963	809	2,996	5.33
All passenger vehicles	209,842	73,612	283,454	265,729	549,183	57,662	9,524	38,904	675	14.12
Trucks and combinations	55,355	18,775	74,130	41,340	115,470	11,159	10,348	14,514	1,301	7.96
All motor vehicles	265,197	92,387	357,584	307,069	664,653	68,821	9,658	53,418	776	12.44

[1] Includes taxicabs and light trailer combinations pulled by passenger cars.
[2] Bus registration adjusted for estimated additional non-revenue buses included with commercial bus registrations.

Trucks and combinations accounted for 16 percent of the vehicles and 17 percent of the travel. The average truck or combination traveled 10,348 miles in 1958, or about 9 percent more than the average passenger car; but it consumed twice as much fuel, 1,301 gallons, at a rate of 7.96 miles per gallon. These averages for trucks and combinations are almost identical with those for 1957.

The average truck or combination traveled 55,355 million vehicle-miles on main rural roads in 1958, or about 48 percent of all travel by this vehicle type, whereas 38 percent of the passenger car travel was on main rural roads. The 1958 truck travel represents an increase of only 0.2 percent on these highways.

Buses, which accounted for the remaining 1 percent of the vehicles and 1 percent of the travel, experienced an actual decrease in total travel during 1958 despite an increase of 3 percent in school and nonrevenue bus travel. A decrease of more than 4 percent in commercial bus travel more than outweighed the school bus travel increase.

Common-Carrier Passenger and Freight Services Available to Communities on the Interstate Highway System

HIGHWAY COST ALLOCATION STUDY
OFFICE OF RESEARCH
BUREAU OF PUBLIC ROADS

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The National System of Interstate and Defense Highways will undoubtedly permit increased and more extensive common-carrier highway freight hauling and intercity bus services, thereby perhaps influencing the availability of alternative modes of common-carrier transportation and the distribution of traffic among the several competing agencies. This article reports on a study made to determine the current situation with regard to the number of communities located on the Interstate System and the availability of the several forms of common-carrier passenger and freight services to them.

IN ORDER to assess the service potentialities of the National System of Interstate and Defense Highways in comparison with parallel services offered by other transportation media, a brief survey was conducted by the Bureau of Public Roads in 1958 to determine the number of communities served by the system and the types of common-carrier passenger and freight transportation service facilities available to them.

The study did not obtain information as to the quality or quantity of the available services; it was limited simply to the availability of such service. Common-carrier passenger service included that by highway, railway, airway, and waterway; common-carrier freight service included the same four modes of transportation and also service by crude petroleum pipeline and petroleum product pipeline.

Trends in Freight and Passenger Services

Before reporting the study and the information collected by it, a brief discussion of intercity passenger and freight movement will be useful in establishing the scope and nature of transportation in the United States as a whole.[1] The importance of the Interstate System in the picture is evidenced by the forecast that by 1971 this 41,000-mile system, comprising little more than 1 percent of all road and street mileage in the nation, will be carrying almost 21 percent of all motor-vehicle travel.

In 1956, on the highways of the Nation, an estimated 253.8 billion ton-miles of cargo, representing 19 percent of the Nation's 1,360.1 billion ton-miles of intercity freight hauling, were transported by truck. Highway freight hauling had increased to nearly five times the 1939 level (52.8 billion ton-miles), when less than 10 percent of the total intercity freight was carried by this method.

The growth in highway passenger travel, essentially attributed to the automobile, has been the major contributor in recent years to the increase in total passenger travel. From 1949 through 1956, total intercity passenger travel by all modes of transportation increased 55 percent, from 450.2 to 698.9 billion passenger-miles. Automobile travel increased 64 percent, from 376.3 to 617.7 billion passenger-miles, whereas total common-carrier passenger travel increased only 10 percent, from 73.9 to 81.2 billion passenger-miles. Thus, automobile travel accounted for 97 percent of the increase in total intercity passenger travel during this period. Concurrently, intercity bus travel declined nearly 10 percent, from 27.9 to 25.2 billion passenger-miles. As a result of the upsurge in automobile travel during the 8-year period, total highway passenger travel (automobile and bus combined) increased 59 percent, and its share of total intercity passenger travel increased from 89.8 to 92.0 percent.

By 1980, the population of the United States is expected to be at least 245 million and the

Figure 1.—The National System of Interstate and Defense Highways, December 1957.

[1] Data on ton-miles of freight carried and passenger-miles traveled are taken from the *72d Annual Report of the Interstate Commerce Commission, Fiscal Year 1958*, pp. 9–15, and Statements Nos. 568 and 580 of the Interstate Commerce Commission.

gross national product, expressed in today's purchasing power, is estimated to approach $900 billion. An extrapolation of recent transportation trends also shows that annual intercity freight hauling may exceed 3 trillion ton-miles and intercity passenger travel may approximate 1.8 trillion passenger-miles. Thus, the prospect for the future is for an expanding economy and greater demand for the movement of goods and people; consequently, increasing demands will be placed on the highway system.

Study Procedure

In January 1958, the Bureau of Public Roads field offices were requested to report the number of communities located on the Interstate System as designated on December 31, 1957. Communities were defined as incorporated places with a population of at least 1,000, according to the 1950 census. In addition, since legislation governing Federal aid for highways defines urban areas as municipalities or other urban places having a population of 5,000 or more, the study definition of communities also included all unincorporated places with 5,000 or more inhabitants. Communities were to be grouped in accordance with standard population classes used by the U.S. Bureau of the Census.[2]

To determine which communities were located on the Interstate System, a 10-mile-wide strip or corridor was used as the criterion by the field offices. The midpoint of the corridor was to approximate the location of the Interstate System. If the location had not been approved as of December 31, 1957, the tentative location or projected location was to be used, in that order of preference. A community was considered to be located on the Interstate System if any part of its area fell within the corridor.

Information was also requested as to the types of common-carrier freight and passenger services available to each of the communities. Highway and rail passenger and freight services were considered available to a community if common-carrier stations or loading facilities were located within the incorporated limits of the community, and if an official timetable or other recognition of commitments for service was provided by the carrier or carriers. Schedules of carriers and records of State regulatory agencies aided in determining the availability of service. Service was considered available to all communities located in a metropolitan complex if the carrier or carriers provided service to any part of the metropolitan area.

Air service was considered available to a community if licensed air carriers made scheduled use, for the purpose of accepting or discharging passengers or freight, of airport facilities located not more than 20 miles from any point of the incorporated or urban area boundary.

[2] It might seem that the number of communities for the New England and Middle Atlantic divisions are overstated in this study in comparison with the number of communities reported by the Bureau of the Census. The reason for this is that in some instances data reported for a State may include as communities two or more contiguous places which, in the Census reports, are considered as a single place.

Figure 2.—Routes of major railroads, 1957.

Figure 3.—Routes of scheduled airlines, 1957.

Water service availability was based on two premises: That facilities for dockage were available to vessels engaged in passenger or freight transportation on rivers, other inland waterways, or in coastal service; and that the facilities were within 5 miles of the closest point of the incorporated or urban area boundary.

Pipeline service was considered available if facilities for terminal reception or distribution of crude petroleum or petroleum products (exclusive of natural gas) served a given community directly. Direct service did not include the use of line-haul motor carriers, tank cars, or tankers to effect final distribution.

In determining the availabilities of the various forms of passenger and freight services, Bureau field office personnel were given considerable leeway in interpreting instructions for the study. This, of course, was necessary in order to make a realistic appraisal of services, particularly in the smaller communities. As a result, the data included in appendix tables A and B (pp. 280 281) are probably not strictly comparable on a State-by-State basis, but are as nearly so as is possible in a survey of this nature.

Transportation Networks

The general location of the Interstate System is shown in figure 1. Figure 2 depicts the networks of the major railroads, and figure 3, the routes of certified trunkline air carriers and those of the local service air carriers. In comparing these routes of highway, rail, and air transportation, it is immediately evident that considerable paralleling of services exists, and that the main routes of commerce and the heavily populated areas are well served by the three modes of transport.

Since pipeline and waterway facilities tend to be restricted, in the one case by source of

Table 1.—Number and percentage of communities in the United States located on the Interstate System and their estimated populations, classified by population group

Population group	Total number of communities in the U.S.[1]	Communities on the Interstate System		Population of all communities in the U.S.[1]	Estimated population of communities on Interstate System [2]
		Number	Percent of total communities		
1,000–2,500	3,408	1,055	31.0	5,382,637	1,668,617
2,500–5,000	1,557	658	42.3	5,512,970	2,331,986
5,000–10,000	1,176	706	60.0	8,138,596	4,883,158
10,000–25,000	778	526	67.6	11,866,505	8,021,757
25,000–50,000	252	216	85.7	8,807,721	7,548,217
50,000–100,000	126	110	87.3	8,930,823	7,796,608
100,000–250,000	65	65	100.0	9,478,662	9,478,662
250,000–500,000	23	23	100.0	8,241,560	8,241,560
500,000–1,000,000	13	13	100.0	9,186,945	9,186,945
Over 1,000,000	5	5	100.0	17,404,450	17,404,450
All communities	7,403	3,377	45.6	92,950,869	76,561,960

[1] Census of Population: 1950, vol. I, table K, p. xxxii. For purposes of this study, incorporated places of 1,000 or more population, and unincorporated places with 5,000 or more population are referred to as communities.

[2] The study did not obtain data on the population of communities on the Interstate System. The estimates were derived, for each population group, by using the percentage relationship of communities on the System to total communities, applied against the total population.

product and in the other case by geography, they are not illustrated. In spite of the fact that pipelines are heavily concentrated in the West South Central and West North Central States, they do serve as distributors of crude petroleum and petroleum products to a considerable number of communities along the Interstate System. This is particularly evident in Illinois and Ohio. Navigable waterways provide many areas of the eastern half of the United States and the Pacific Coast States with good transportation service.

Communities Served by the Interstate System

The total number of communities in the United States, by population group, are compared in table 1 with the number of communities served by the Interstate System as of December 31, 1957. A State-by-State compilation of the number of communities served by the Interstate System, by population group, is provided in appendix table A. As previously defined, the term "community" refers to incorporated places with 1,000 or more population and unincorporated places with 5,000 or more population, according to the 1950 census.

Also presented in table 1 are the percentages of all communities in each population group that were served by the Interstate System, the aggregate population of all communities in each population group, and the estimated population [3] of communities in each population group that were served by the Interstate System. Of particular significance is the fact that over four-fifths of the people in all communities of the United States were served by the Interstate System.

At the time of the 1950 census, the number of communities in the United States with populations of 5,000 and over was 2,438; of these, 1,664 were served by the Interstate System. Similarly, of the 4,965 communities in the 1,000 to 5,000 population range, 1,713 were served. Thus, 3,377 communities or nearly 46 percent of all communities were located within the Interstate System corridor established for this study. All cities of at least 100,000 population, 93 percent of all cities with 50,000 population and over, 89 percent of all cities with 25,000 population and over, or 76 percent of all cities with 10,000 population and over were served by the Interstate System.

Common-Carrier Passenger Services

A distribution of the types of common-carrier passenger services available to the 3,377 communities located on the Interstate System is presented in table 2. Approximately 99 percent (all but 16 communities) were served by at least one of the common-carrier passenger services—highway, rail, air, or water. The 16 communities not having common-carrier service in 1957 were in the two smallest population groups.

Bus service was the most prevalent type of common-carrier passenger transportation available to communities on the Interstate System. In general, the study shows that such service was available to almost all communities on the system, even the smallest. This statement can be given even wider application when considering all incorporated and unincorporated places, regardless of population and location with respect to the Interstate System. It has been estimated that the only intercity common-carrier passenger transportation available to 40,000 communities in the United States is bus service.[4]

Although the availability of each form of common-carrier passenger service diminished in the smaller communities located on the Interstate System, the availability of bus service diminished least. Among the 1,040 communities in the 1,000–2,500 population group having common-carrier passenger service in 1957, 92 percent had bus service, 73 percent had rail service, 54 percent had air service, and 11 percent had water service.

As expected, many of the communities on the Interstate System had more than one type of common-carrier passenger service. All four forms were available in the five cities with over 1 million population, and with each progressively smaller population group, the average number of services available declined from 3.69 to 2.30. For all population groups, the number of services averaged 2.66 per community.

Geographical distribution of passenger service

Availability of common-carrier passenger service to communities grouped according to census divisions is shown in table 3. Similar information on a State-by-State basis is presented in appendix table B. Bus transporta-

[3] See footnote 2, table 1.

[4] *Bus Facts*, National Association of Motor Bus Operators, 27th ed., 1958, p. 6.

Table 2.—Availability of each mode of common-carrier passenger service to communities on the Interstate System, by population group

Population group	Total communities on Interstate System	Number of communities having common-carrier passenger service [1]	Number and percentage of communities having indicated common-carrier passenger service available								Total passenger services available	Ratio: total passenger services/total communities
			Highway (bus)		Rail		Air		Water			
			Number	Percent	Number	Percent	Number	Percent	Number	Percent		
1,000–2,500	1,055	1,040	961	92.4	763	73.4	560	53.8	110	10.6	2,394	2.30
2,500–5,000	658	657	630	95.9	509	77.5	432	65.8	112	17.0	1,683	2.56
5,000–10,000	706	706	691	97.9	600	85.0	488	69.1	150	21.2	1,929	2.73
10,000–25,000	526	526	521	99.0	471	89.5	417	79.3	134	25.5	1,543	2.93
25,000–50,000	216	216	215	99.5	206	95.4	200	92.6	48	22.2	669	3.10
50,000–100,000	110	110	108	98.2	106	96.4	107	97.3	27	24.5	348	3.16
100,000–250,000	65	65	65	100.0	65	100.0	65	100.0	19	29.2	214	3.29
250,000–500,000	23	23	23	100.0	23	100.0	23	100.0	8	34.8	77	3.35
500,000–1,000,000	13	13	13	100.0	13	100.0	13	100.0	9	69.2	48	3.69
Over 1,000,000	5	5	5	100.0	5	100.0	5	100.0	5	100.0	20	4.00
All communities	3,377	3,361	3,232	96.2	2,761	82.1	2,310	68.7	622	18.5	8,925	2.66

[1] 16 communities did not have common-carrier passenger service: 15 in the 1,000–2,500 population group, and 1 in the 2,500–5,000 population group.

Table 3.—Availability of each mode of common-carrier passenger service to communities on the Interstate System, by census division

Census division	Total communities on Interstate System [1]	Number and percentage of communities having indicated common-carrier passenger service available [2]							
		Highway (bus)		Rail		Air		Water	
		Number	Percent	Number	Percent	Number	Percent	Number	Percent
New England	389	376	96.7	252	64.8	369	94.9	33	8.5
Middle Atlantic	792	740	94.8	614	78.6	637	81.6	275	35.2
South Atlantic (North)	122	120	99.2	101	83.5	73	60.3	26	21.5
South Atlantic (South)	247	247	100.0	214	86.6	151	61.1	31	12.6
East North Central	720	653	91.1	608	84.8	479	66.8	217	30.3
East South Central	175	174	99.4	151	86.3	90	51.4	3	1.7
West North Central	282	273	96.8	249	88.3	129	45.7	----	----
West South Central	243	243	100.0	231	95.1	114	46.9	1	.4
Mountain	173	173	100.0	151	87.3	84	48.6	----	----
Pacific	234	233	100.0	199	81.5	184	79.0	36	15.5
All census divisions	3,377	3,232	96.2	2,761	82.1	2,310	68.7	622	18.5

[1] 16 communities did not have common-carrier passenger service: 11 in the Middle Atlantic division, 3 in the East North Central division, and 1 each in the South Atlantic (North) and Pacific divisions.
[2] Percentages relate to the number of communities having passenger services.

tion was available to all communities on the Interstate System having common-carrier passenger service in the South Atlantic (South), West South Central, Mountain, and Pacific census divisions. In the remaining six divisions, 91 to 99 percent of such communities were provided with bus service.

Intercity rail passenger transportation was offered extensively across the nation to communities on the Interstate System. The extent of such service ranged from 65 percent of the communities having common-carrier passenger service in the New England division to 95 percent of the communities in the West South Central division.

Air passenger service was relatively more available to communities on the Interstate System in New England than in other areas of the Nation. Ninety-five percent of the 389 communities located on the system in this census division were provided with air passenger service. A possible explanation for the high percentage might be that a comparatively larger proportion of communities in the New England division fell within the study corridor because of the limited area involved and the high density of population. By comparison, less than half of the communities on the Interstate System in the West North Central, West South Central, and Mountain divisions had air passenger service.

Passenger service by water was negligible in all areas of the country except for the Middle Atlantic and East North Central census divisions, where approximately one-third of the communities on the Interstate System having common-carrier passenger transportation were provided this service.

Common-Carrier Freight Services

All communities located on the Interstate System had one or more of the five common-carrier freight services: highway, rail, air, water, or pipeline. Highways provided common-carrier freight service to more communities than any other form of transportation. Table 4 shows that trucking service was available to 3,345 communities, or 99 percent of all communities located on the Interstate System. In comparison, intercity bus service was available to 96 percent of the communities.

Rail freight service was available to 94 percent of all communities on the Interstate System. This service applied to all communities over 25,000 population and to 93 percent of the communities under 25,000 population. As would be expected, the

Table 4.—Availability of each mode of common-carrier freight service to communities on the Interstate System, by population group

Population group	Total communities on Interstate System [1]	Number and percentage of communities having indicated common-carrier freight service available												Total freight services available	Ratio: total freight services/total communities
		Highway (truck)		Rail		Air		Water		Pipeline					
										Crude petroleum		Petroleum products			
		Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent		
1,000–2,500	1,055	1,031	97.7	940	89.1	555	52.6	200	19.0	68	6.4	130	12.3	2,924	2.77
2,500–5,000	658	653	99.2	616	93.6	427	64.9	186	28.3	43	6.5	87	13.2	2,012	3.06
5,000–10,000	706	703	99.6	679	96.2	485	68.7	246	34.8	49	6.9	99	14.0	2,261	3.20
10,000–25,000	526	526	100.0	510	97.0	412	78.3	212	40.3	55	10.5	85	16.2	1,800	3.42
25,000–50,000	216	216	100.0	216	100.0	198	91.7	93	43.1	22	10.2	39	18.1	784	3.63
50,000–100,000	110	110	100.0	110	100.0	104	94.5	51	46.4	14	12.7	29	26.4	418	3.80
100,000–250,000	65	65	100.0	65	100.0	64	98.5	38	58.5	10	15.4	20	30.8	262	4.03
250,000–500,000	23	23	100.0	23	100.0	23	100.0	14	60.9	10	43.5	13	56.5	106	4.61
500,000–1,000,000	13	13	100.0	13	100.0	13	100.0	13	100.0	7	53.8	7	53.8	66	5.08
Over 1,000,000	5	5	100.0	5	100.0	5	100.0	5	100.0	3	60.0	4	80.0	27	5.40
All communities	3,377	3,345	99.1	3,177	94.1	2,286	67.7	1,058	31.3	281	8.3	513	15.2	10,660	3.16

[1] All communities on the Interstate System had one or more common-carrier freight services.

Table 5.—Availability of each mode of common-carrier freight service to communities on the Interstate System, by census division

Census division	Total communities on Interstate System	Number and percentage of communities having indicated common-carrier freight service available											
		Highway (truck)		Rail		Air		Water		Pipeline			
										Crude petroleum		Petroleum products	
		Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Number	Percent
New England	389	389	100.0	343	88.2	369	94.9	53	13.6	2	0.5	18	4.6
Middle Atlantic	792	790	99.7	745	94.1	637	80.4	456	57.6	16	2.0	31	3.9
South Atlantic (North)	122	120	98.4	113	92.6	56	45.9	46	37.7	1	.8	1	.8
South Atlantic (South)	247	243	98.4	244	98.8	151	61.1	39	15.8	----	----	52	21.1
East North Central	720	713	99.0	672	93.3	479	66.5	267	37.1	113	15.7	230	31.9
East South Central	175	174	99.4	164	93.7	88	50.3	40	22.9	6	3.4	6	3.1
West North Central	282	282	100.0	278	98.6	125	44.3	53	18.8	48	17.0	63	22.3
West South Central	243	227	93.4	236	97.1	114	46.9	24	9.9	63	25.9	62	25.5
Mountain	173	173	100.0	157	90.8	84	48.6	----	----	31	17.9	41	23.7
Pacific	234	234	100.0	225	96.2	183	78.2	80	34.2	1	.4	9	3.8
All census divisions	3,377	3,345	99.1	3,177	94.1	2,286	67.7	1,058	31.3	281	8.3	513	15.2

smallest communities were most dependent on highway transportation as indicated by the fact that 98 percent of the communities in the 1,000–2,500 population group had truck service whereas 89 percent had rail service.

At least 9 out of 10 communities had rail freight service; 9 out of 10 had truck service; slightly over two-thirds (2,286) were provided air freight service; about one-third (1,058) had access to freight shipping on domestic waterways. Approximately one-fifth of the communities were directly served by crude petroleum and/or petroleum products pipeline service, but no attempt was made to determine the number of communities having both services.

The large cities had both air and water common-carrier freight services, although the availability of air service was more extensive. The volume of air freight in ton-miles is small, however, in comparison with the volume of freight in ton-miles carried by water, the ratio being 1 to 386 in 1957.[5] All cities with 250,000 or more population were provided air freight service, but only 78 percent of the 41 cities within this group were provided water freight service. The proportion of communities served by air and/or water freight carriers decreased rapidly, however, in descending community population groups. In the lowest population group, about 53 percent of the communities on the Interstate System were served by air freight carriers, whereas only 19 percent were served by common carriers operating on the domestic waterways.

The restriction of pipeline facilities to one commodity group accounts for the limited availability of this mode of transportation to communities on the Interstate System. These facilities were available to four of the five cities with populations exceeding 1 million; New York City alone in this class did not have direct pipeline service. In the medium- and small-sized communities (below 50,000 population), only a limited number had pipeline service. Eight percent or only 281 communities of the 3,377 located on the Interstate System had facilities for terminal reception or distribution of crude petroleum, and 15 percent or 513 communities had facilities for terminal reception or distribution of petroleum products.

A combined total of 10,660 freight services were available to the 3,377 communities on the Interstate System, the average being 3.16 freight services per community. Excluding pipelines, the number of freight services offered per community becomes 2.92.[9] This may be compared with 2.66 common-carrier passenger services available per community.

[5] See footnote 1, p. 276.

Table A.—Number of communities [1] on the Interstate System by census divisions, States, and population groups

Census division and State	Total communities on Interstate System	Population group, 1950 census									
		1,000–2,500	2,500–5,000	5,000–10,000	10,000–25,000	25,000–50,000	50,000–100,000	100,000–250,000	250,000–500,000	500,000–1,000,000	Over 1,000,000
United States	3,377	1,055	658	706	526	216	110	65	23	13	5
New England	389	83	86	78	77	36	17	11		1	
Connecticut	96	22	21	17	15	15	2	4			
Maine	19	2	3	6	6	1	1				
Massachusetts	201	38	49	38	43	16	10	6		1	
New Hampshire	20	10	5	2	1	1	1				
Rhode Island	31	3	4	8	10	2	3	1			
Vermont	22	8	4	7	2	1					
Middle Atlantic	792	206	162	209	136	11	19	12	3	2	2
New Jersey	215	36	38	62	46	20	7	4	2		
New York	244	68	53	57	40	13	6	4	1	1	1
Pennsylvania	333	102	71	90	50	8	6	4		1	1
South Atlantic (North)	122	45	17	25	16	6	7	4		2	
Delaware	6	2		3				1			
District of Columbia	1									1	
Maryland	36	18	6	5	5	1				1	
Virginia	50	11	6	14	8	4	4	3			
West Virginia	29	14	5	3	3	1	3				
South Atlantic (South)	247	89	52	42	32	15	11	5	1		
Florida	81	21	22	14	11	8	2	3			
Georgia	70	30	17	8	10	1	2	1	1		
North Carolina	61	25	6	11	9	5	4	1			
South Carolina	35	13	7	9	2	1	3				
East North Central	720	261	139	127	99	50	26	9	4	3	2
Illinois	251	97	51	50	31	12	8	1			1
Indiana	87	30	16	16	10	8	3	3	1		
Michigan	121	45	15	15	25	11	7	2			1
Ohio	205	71	45	37	26	13	5	3	3	2	
Wisconsin	56	18	12	9	7	6	3			1	
East South Central	175	60	42	31	19	11	4	5	3		
Alabama	49	17	9	10	6	3	1	2	1		
Kentucky	46	16	12	9	4	2	2		1		
Mississippi	39	15	11	4	3	5	1				
Tennessee	41	12	10	8	6	1		3	1		
West North Central	282	119	56	41	34	13	7	4	3	2	
Iowa	35	20	5	3	2	2	2	1			
Kansas	31	11	4	6	6	1	1	2			
Minnesota	72	31	13	12	10	3		1	1	1	
Missouri	92	34	25	15	10	4	2		1	1	
Nebraska	24	8	7	4	3		1		1		
North Dakota	15	8		3	2	2					
South Dakota	13	7	2	1	1	1					
West South Central	243	84	38	57	36	10	5	8	3	2	
Arkansas	28	8	3	12	2	2		1			
Louisiana	37	13	7	8	3	3		2		1	
Oklahoma	44	12	10	12	7	1		2			
Texas	134	51	18	25	24	4	5	3	3	1	
Mountain	173	63	37	36	20	11	3	2	1		
Arizona	21	7	6	6		1		1			
Colorado	23	9	2	5	4	1	1		1		
Idaho	23	7	7	2	5	2					
Montana	23	10	4	3	3	3					
Nevada	9	2	2	3	1	1					
New Mexico	20	6	2	7	3	1	1				
Utah	40	18	10	9		1	1	1			
Wyoming	14	4	4	1	4	1					
Pacific	234	45	29	57	57	23	11	5	5	1	1
California	161	19	13	43	49	18	11	3	3	1	1
Oregon	32	9	8	8	4	2			1		
Washington	41	17	8	6	4	3		2	1		

[1] Includes all incorporated places with populations of 1,000 and over, and all unincorporated places with populations of 5,000 and over.

Freight services offered per community ranged from 2.77 for communities under 2,500 population to 5.40 for cities exceeding 1 million population. The number of services provided the average community are based on six modes of transportation rather than five because of the two categories of pipelines.

Geographical distribution of freight services

The various forms of freight service available by geographic areas and by States are shown in table 5 and appendix table B. A greater number of communities on the Interstate System were served by truck common carriers than by any other form of freight transportation in all areas of the country except the South Atlantic (South) and the West South Central census divisions where rail freight service was more extensive. All communities in the New England, West North Central, Mountain, and Pacific divisions had truck service, and in the remaining six divisions, truck service was available to at least 93 percent of the communities.

Ninety-nine percent of the communities in the South Atlantic (South) and West North Central census divisions had rail freight service; at the other extreme, 88 percent of the communities in the New England division were provided such service.

Air freight service, which was available to about two-thirds of the communities on the Interstate System, was offered to a greater proportion of communities in the New England census division than in any other area of the country. Ninety-five percent of the communities in New England had such service, and following in order were the Middle Atlantic and Pacific divisions with 80 and 78 percent, respectively. Lowest on the scale were the West North Central and South Atlantic (North) divisions where air freight serviced 44 and 46 percent of the communities.

Only about one in three communities on the Interstate System had access to water freight service. The Middle Atlantic census division ranked highest with nearly 58 percent of the communities having such service. Next in order were the South Atlantic (North) and East North Central divisions with 38 and 37 percent of the communities so served.

Table B.—Availability of each mode of common-carrier transportation service to communities on the Interstate System, by census divisions and by States

Census division and State	Total communities on Interstate System	Number of communities having indicated passenger service				Total passenger services available	Number of communities having indicated freight service						Total freight services available
		Highway (bus)	Rail	Air	Water		Highway (truck)	Rail	Air	Water	Pipeline		
											Crude petroleum	Petroleum products	
United States	3,377	3,232	2,761	2,310	622	8,925	3,345	3,177	2,286	1,058	281	513	10,660
New England	389	376	252	369	33	1,030	389	343	369	53	2	18	1,174
Connecticut	96	87	59	89	2	237	96	82	89	3			270
Maine	19	19	19	17	2	57	19	19	17	10	2	9	76
Massachusetts	201	198	124	201	12	535	201	174	201	16		8	600
New Hampshire	20	19	14	14		47	20	16	14	1			51
Rhode Island	31	31	15	31	14	91	31	30	31	21		1	114
Vermont	22	22	21	17	3	63	22	22	17	2			63
Middle Atlantic	792	740	614	637	275	2,266	790	745	637	456	16	31	2,675
New Jersey	215	210	155	162	158	685	214	195	162	162	5	4	742
New York	[1] 244	213	218	205	95	731	244	240	205	192	8	9	898
Pennsylvania	[1] 333	317	241	270	22	850	332	310	270	102	3	18	1,035
South Atlantic (North)	122	120	101	73	26	320	120	113	56	46	1	1	337
Delaware	6	6	6	6		18	4	6	6	5			21
District of Columbia	1	1	1	1	1	4	1	1	1	1	1	1	6
Maryland	[1] 36	34	32	30	24	120	36	35	30	26			127
Virginia	50	50	43	19	1	113	50	49	19	14			132
West Virginia	29	29	19	17		65	29	22					51
South Atlantic (South)	247	247	214	151	31	643	243	241	151	39		52	729
Florida	81	81	78	61	27	247	77	81	61	30			249
Georgia	70	70	60	38	3	171	70	69	38	6		42	225
North Carolina	61	61	50	31		142	61	60	31	1			153
South Carolina	35	35	26	21	1	83	35	34	21	2		10	102
East North Central	720	653	608	479	217	1,957	713	672	479	267	113	230	2,474
Illinois	[1] 251	249	210	166	83	708	251	237	166	127	33	108	922
Indiana	[1] 87	85	54	48	12	199	87	85	48	12	8	32	272
Michigan	121	97	101	87	33	318	120	99	89	33	8	7	356
Ohio	205	170	196	141	72	579	199	196	139	72	63	77	746
Wisconsin	56	52	47	37	17	153	56	55	37	23	1	6	178
East South Central	175	174	151	90	3	418	174	164	88	40	6	6	478
Alabama	49	48	43	24	3	118	48	47	24	3	3	3	128
Kentucky	46	46	44	31		121	46	45	31	21	2		145
Mississippi	39	39	38	18		95	39	39	18	7	1	1	105
Tennessee	41	41	26	17		84	41	33	15	9		2	100
West North Central	282	273	249	129		651	282	278	125	53	48	63	849
Iowa	35	35	27	17		79	35	35	17	6	7	7	107
Kansas	31	30	31	12		73	31	31	12	5	16	16	111
Minnesota	72	72	65	40		177	72	72	35	29	24	27	259
Missouri	92	85	82	39		206	92	89	39	9		2	231
Nebraska	24	24	22	8		54	24	24	8	4		4	64
North Dakota	15	15	15	6		36	15	15	7		1	6	44
South Dakota	13	12	7	7		26	13	12	7			1	33
West South Central	243	243	231	114	1	589	227	236	114	24	63	62	726
Arkansas	28	28	28	7		63	20	28	7	1		1	57
Louisiana	37	37	36	18	1	92	29	37	18	10	6	4	104
Oklahoma	44	44	40	17		101	44	41	17				102
Texas	134	134	127	72		333	134	130	72	13	57	57	463
Mountain	173	173	151	84		408	173	157	84		31	41	486
Arizona	21	21	19	7		47	21	19	7			6	53
Colorado	23	23	22	12		57	23	23	12		8	8	74
Idaho	23	23	23	10		56	23	23	10			3	59
Montana	23	23	22	8		53	23	22	8		15	15	83
Nevada	9	9	8	6		23	9	9	6			2	26
New Mexico	20	20	17	4		41	20	18	4		2	2	46
Utah	40	40	27	31		98	40	30	31		1		102
Wyoming	14	14	13	6		33	14	13	6		5	5	43
Pacific	234	233	190	184	36	643	234	225	183	80	1	9	732
California	161	161	138	136	20	455	161	154	135	46	1	5	502
Oregon	32	32	21	24	7	84	32	32	24	13		2	103
Washington	[1] 41	40	31	24	9	104	41	39	24	21		2	127

[1] Common-carrier passenger services were not available to 16 communities located in the following States: New York, 5 communities; Pennsylvania, 6 communities; Maryland, 1 community; Illinois, 2 communities; Indiana, 1 community; and Washington, 1 community.

Haulage of bulk commodities on the inland waterways has been expanding rapidly in recent years, and with the development of the St. Lawrence Seaway, the tonnage on the Great Lakes and connecting waterways will increase and more communities on the Interstate System will be served, particularly in the East North Central and Middle Atlantic States.

Pipeline service was concentrated in four census divisions: East North Central, West North Central, West South Central, and Mountain. It is in these areas that most of the crude petroleum is produced and refined. They also serve as distributing centers for petroleum products. It is interesting to note that the South Atlantic (South) division, an area which does not have crude petroleum pipelines, serves as a distributing center for petroleum products. This, of course, indicates that crude petroleum is shipped into the area by other than pipeline facilities.

General Comments

Highways provided more extensive common-carrier passenger and freight services than any other medium of transport. Railroads ranked second, and were followed in order by airlines and waterways.

Of the 3,377 communities located on the Interstate System, 72 percent or 2,445 communities were located in States east of the Mississippi River. These States, which comprise 29 percent of the land area of the continental United States and 68 percent of the population, make up six census divisions: New England, Middle Atlantic, South Atlantic (North), South Atlantic (South), East North Central, and East South Central.

Nearly one-half of the Interstate System mileage is located in the six census divisions just enumerated. On this basis, there was an average of one community for each 8-mile length of the system. In the remaining four census divisions to the west of the Mississippi River, there were 932 communities located on the Interstate System, or an average of 1 community for each 22-mile length of the system.

The development of the Interstate System has had and should continue to have a significant effect on the quality and quantity of highway transport services offered to these communities, upon coordination of transportation services, and upon competition among the several modes of transportation.

Forecasts of Population, Motor-Vehicle Registrations, Travel, Fuel Consumption

(*Continued from page 274*)

The miles-per-gallon rates for the census divisions (table 11) showed only minor variations in 1956. The Middle Atlantic division reported the highest at 13.0 and the West South Central the lowest, 11.6, a variation of only 12 percent. The forecast values for 1976, however, present a different picture. A variation of 20 percent is predicted, ranging from 13.6 miles per gallon in the South Atlantic (North) division to 11.3 in the West South Central division. During the forecast period, three divisions expect increases from 0.04 to 0.70 miles per gallon, while the remaining seven anticipate decreases ranging from 0.01 to 0.46. It would appear that many of the States, aware of the several variables that must be considered in making forecasts of motor-fuel consumption and resulting revenues did not wish to introduce still another variable, that of changing miles-per-gallon values, into their forecasts. This can be considered usually as a prudent approach. The principal justification of the practice of using a fairly constant miles-per-gallon value in preparing forecasts is one of neutrality in the subsequent forecasts of revenues as a function of vehicle-miles traveled. A prediction of increased productivity per vehicle-mile through a lessened rate of fuel consumption results in introducing an extra, and possible unnecessary, variable into the forecasting procedures.

Comments on Forecasting Procedures

A review of the forecasts of travel and needs made in the past shows that, in practically all cases, the forecasts have fallen woefully short of reality. It is highly possible that such estimates were based on inadequate data, and the resulting needs and travel estimates were inevitably bound to be too low because the basic data were also too low. There has also occurred a series of events, within the period of time in which the development of highways has become so important in the American way of life, that has had a tendency to obscure the trends or at least introduce uncertainties into forecasting travel and needs. These events are well known to all—the depression of the 1930's, World War II, and the tremendous increase in travel and registrations accompanying the general economic expansion of the last decade. The forecasting of highway use was not the only facet of our future economy which was invariably pitched too low. So were the population forecasts, and the forecasts of gross national product and personal income, all key factors in estimating future highway travel and needs.

It was not until the apparent close relationship between gross national product (GNP) and total travel was observed that forecasts of travel were projected at a level considered as being realistic. There is reason to believe that this historic close relationship has led to a tendency to extend it into the future—to tie traffic forecasts rather closely to projections of GNP. In view of developments of the last decade, this procedure, which disregards the changing composition of the GNP, could quite possibly result in a too-conservative forecast of travel. Investigations of the trend growth in the two series since 1950 show that total travel is increasing at a more rapid rate than GNP. Whether it will continue to increase, relatively, is problematical, but the most conservative extension of the 1950–58 trend would result in a 1976 travel estimate considerably higher than the one developed in this report.

A review of the information submitted by the States shows that, in general, they did an excellent job in preparing their forecasts, although having limited data available in some areas. Probably the most critical areas in which background data were lacking were the classification of travel by rural-urban areas, the projected growth of metropolitan areas and their attendant traffic problems, and projections of economic and population growth in the States. All of the items mentioned have an important bearing on travel and highway needs. A dearth or absence of adequate information in these areas makes the task of projecting highway travel and needs difficult and its evaluation doubly so.

Because of rapidly changing events and technology, and because of the behavioral nature of many of the factors involved, no one can oraculate with finality about our future population, motor-vehicle registrations, highway traffic, and highway needs. There is, nevertheless, much to be done in this field of forecasting highway use and needs. The development of more accurate and adequate forecasting techniques would result in projections that could be used with greater assurance by highway administrators than those they now have available.

Surface and Subsurface Temperature Variations and Comparisons

Reported by HAROLD L. BOEN and GERARD A. DeMARRAIS
United States Weather Bureau

There is a need for correlation of surface and subsurface temperature variations with moisture conditions and the performance of highway pavements, base courses, and subgrades. Information is also needed to aid in accurate predictions of frost penetration and subsurface temperature variations from weather reports. The gradual collection of such data for various parts of the country would be very helpful to highway engineers. This article presents temperature variation data collected in Idaho by the U.S. Weather Bureau.

THE U.S. Weather Bureau Office of Idaho Falls, Idaho, in the course of a series of applied meteorological studies,[1] conducted investigations concerned with soil-surface and subsurface temperature variations and comparisons which are of interest to highway engineers. The soil-surface study contained the quantitative results of temperature variations near the surface over a 4-year period. The 2-year subsurface temperature investigation dealt with a comparison of temperature to a depth of 7 feet beneath an asphalt road surface and under a nearby sandy surface.

Site Description and Pertinent Climatology

The temperature observations were taken at the National Reactor Testing Station, 50 miles west of Idaho Falls, Idaho. The station is located on the Snake River Plain which has an average elevation of 5,000 feet and is completely surrounded by mountains. The area has desert-like characteristics, a sandy surface with occasional lava rock outcroppings. Average daily temperatures for the station are somewhat lower than most of the U.S., ranging from 15° to 20° F. in winter to 60° and 70° F. in summer. Precipitation is light, approximately 7.5 inches annually. The ground surface is usually snow-covered in winter and dry the remainder of the year.

Temperature Near the Surface

Temperatures near the surface were obtained by using a copper probe (14 inches long and 1 inch in diameter) containing a thermistor connected to a thermograph recorder. The first year's data were collected with the probe unpainted and indicate temperatures that exposed metallic objects might attain. After the first year, the probe was painted black for 3 years and was representative of temperatures experienced on a blacktop surface such as an asphalt road. Calibration of the instrument showed that it was accurate to within 1° to 2° F. The probe seldom recorded the actual extreme temperature because of the very large lag and because the indicated temperature was an average of the surface area of the probe. The probe was supported one-half inch above the surface, and in that position its temperature was determined by radiation, conduction, and convection.

Table 1 shows the temperature variations of the probe and compares these with the free air temperature taken in a nearby weather instrument shelter at a height of 5 feet. As would be expected, direct exposure of the probe to the sun showed a considerably higher temperature than the shelter thermometer recorded, particularly during the warmer months of the year. Comparisons of the copper- and black-colored thermometer probe temperatures showed that painting the probe black resulted in raising the average daily high by as much as 19° F., while the low temperature generally differed by only small amounts.

Of particular interest are the maximum daily ranges of temperature using the black-

Table 1.—Temperature comparisons and variations of the probe thermometer and shelter thermometer (°F.)

Month	Average daily high temperature		Average daily low temperature		Absolute maximum temperature		Absolute minimum temperature		Average daily temperature range		Maximum daily temperature range [1]
	Probe	Shelter	Probe	Shelter	Probe	Shelter	Probe	Shelter	Probe	Shelter	
A.—Copper-Colored Thermometer Probe on Surface. January 1951–March 1952											
January	39	25	3	0	61	38	−19	−26	36	24	62
February	48	29	7	5	63	42	−20	−26	41	24	69
March	53	35	13	13	78	56	−5	−10	40	22	63
April	80	59	27	29	96	74	14	15	53	30	72
May	99	68	33	38	122	85	22	23	66	30	83
June	103	73	35	40	127	88	24	27	68	33	98
July	120	86	45	50	138	95	35	36	74	36	95
August	111	81	45	48	134	93	33	35	66	33	86
September	111	74	36	38	125	85	22	23	76	36	94
October	81	55	27	27	105	78	13	11	54	28	81
November	61	41	17	16	79	58	0	−2	44	25	60
December	36	25	5	5	49	38	−10	−18	31	20	55
B.—Black-Colored Thermometer Probe on Surface. April 1952–April 1955											
January	47	31	17	11	81	49	−11	−24	30	20	68
February	60	36	15	10	100	53	−7	−21	45	26	73
March	72	42	18	16	108	66	−4	−15	53	25	89
April	88	56	26	27	124	77	8	6	62	29	106
May	107	67	34	35	140	91	18	18	73	32	108
June	112	74	41	42	148	95	26	26	72	32	108
July	133	88	47	49	160	99	32	33	86	39	115
August	128	85	44	46	155	99	27	28	85	39	117
September	121	78	35	36	142	92	19	18	86	42	108
October	100	65	24	25	124	82	8	9	77	40	104
November	75	46	16	16	102	63	−8	−13	59	30	92
December	49	30	9	5	81	42	−9	−18	40	25	66
C.—Thermometer Probe Buried One Inch in the Ground. June 1955–May 1958											
January	27	29	19	5	35	46	5	−31	8	25	20
February	31	35	22	11	59	53	2	−29	8	24	30
March	45	44	28	20	65	63	20	−10	18	24	36
April	63	56	34	28	90	76	26	8	29	28	53
May	86	68	46	39	119	89	32	23	41	29	68
June	105	77	50	44	134	95	35	29	55	33	84
July	118	87	56	50	141	99	42	34	62	39	87
August	117	87	53	48	135	97	37	36	64	39	83
September	100	75	42	37	125	96	29	20	57	39	78
October	72	61	32	27	98	82	19	8	40	34	64
November	39	40	23	14	68	65	6	−2	16	27	43
December	30	35	21	12	42	45	10	−14	8	23	17

[1] Maximum temperature range measured in one day by the probe thermometer.

[1] The work described in this report was supported under contract to the Reactor Development Division, U.S. Atomic Energy Commission.

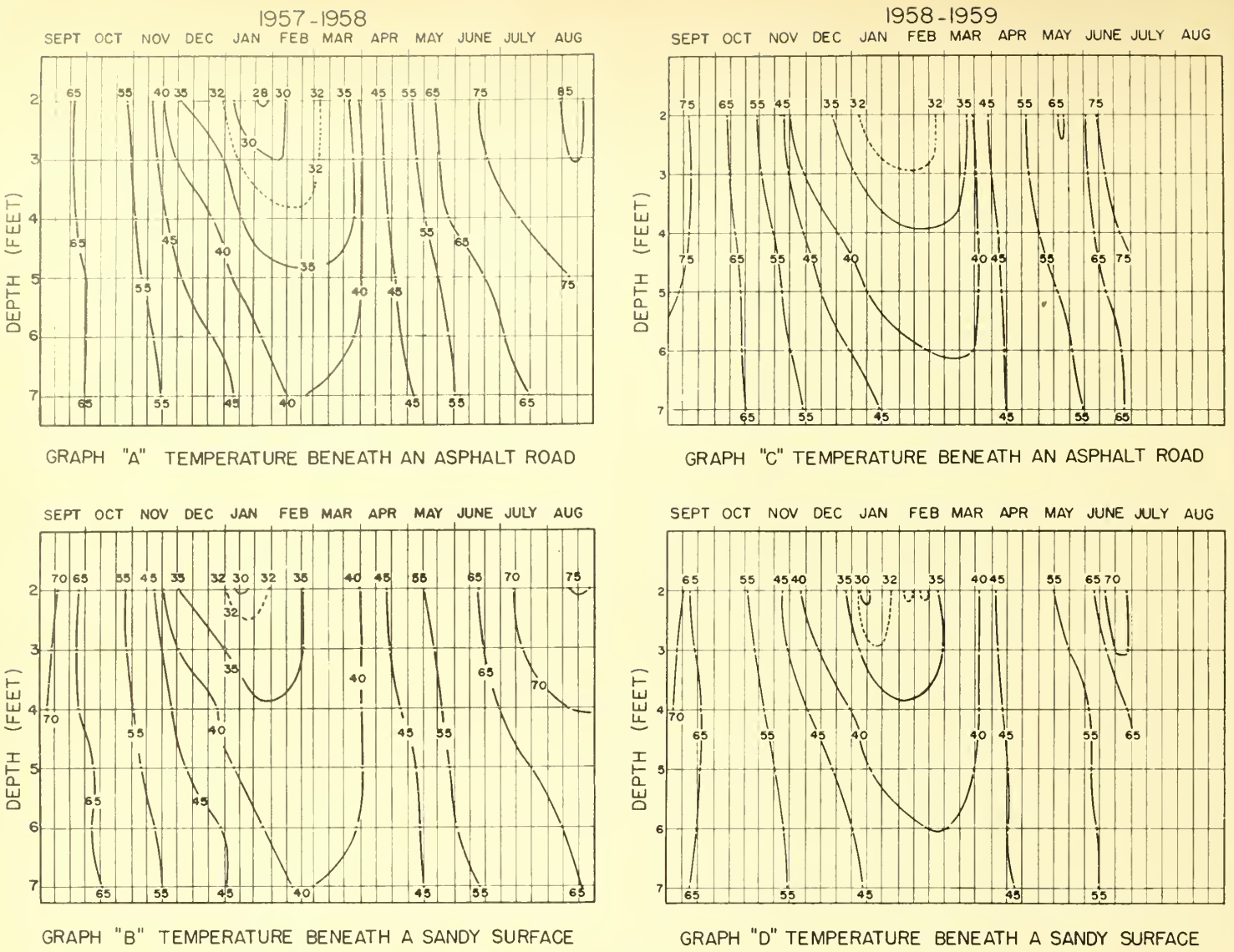
1957-1958
SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG
DEPTH (FEET)
GRAPH "A" TEMPERATURE BENEATH AN ASPHALT ROAD
1958-1959
SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG
DEPTH (FEET)
GRAPH "C" TEMPERATURE BENEATH AN ASPHALT ROAD
SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG
DEPTH (FEET)
GRAPH "B" TEMPERATURE BENEATH A SANDY SURFACE
SEPT OCT NOV DEC JAN FEB MAR APR MAY JUNE JULY AUG
DEPTH (FEET)
GRAPH "D" TEMPERATURE BENEATH A SANDY SURFACE

Figure 1.—A comparison of temperatures recorded at varying depths beneath two types of surface for a 3-year period.

colored probe. Since the probe might be compared to an asphalt surface, the asphalt surface could have daily fluctuations in temperature to over 100° F.

Subsurface Temperatures

The subsurface temperature study, inaugurated in 1957, compared the depth of the freezing level beneath an asphalt road surface to that beneath a sandy surface. Six thermistors were equally spaced at 1-foot depth intervals from 2- to 7-feet and connected to a recorder. One installation was located beneath an asphalt surface and the other installation beneath a nearby sandy surface.

Graphs A–D in figure 1 illustrate temperature profiles for the 2-year period. A comparison of graphs A and B (September 1957–August 1958) showed the freezing level extending to nearly 4 feet under the road surface (graph A), while under the sandy surface (graph B) the 3-foot level remained free of frost during the entire winter. Temperature extremes throughout the first year were greater down to a depth of 4 feet under the road, while below 4 feet the curves in the two graphs compare quite favorably. Graphs C and D, for the second year (September 1958–June 1959), showed the freezing level at nearly the same depth, although the sandy surface (graph D), down through the 2-foot level, showed intermittent periods of thawing. The short period of 30° F. temperatures in January 1959 was attributed to melting snow percolating into the ground and refreezing at air temperatures of near zero. Since both winters were milder than normal the freezing level would be expected to reach a deeper penetration in a normal year.

Editor's note: Highway Research Board Special Reports 18 and 22 concerning the WASHO Road Test contain data on the temperature of air, pavement, base, and subgrade of the test road near Malad, Idaho. The Weather Bureau data reported in this article, also collected in Idaho, are comparable to those reported at the test road site.

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